CONCRETE AND CONSTRUCTIONAL ENGINEERING

INCLUDING PRESTRESSED CONCRETE

NOVEMBER 1961 VOL. LVI. NO. 11

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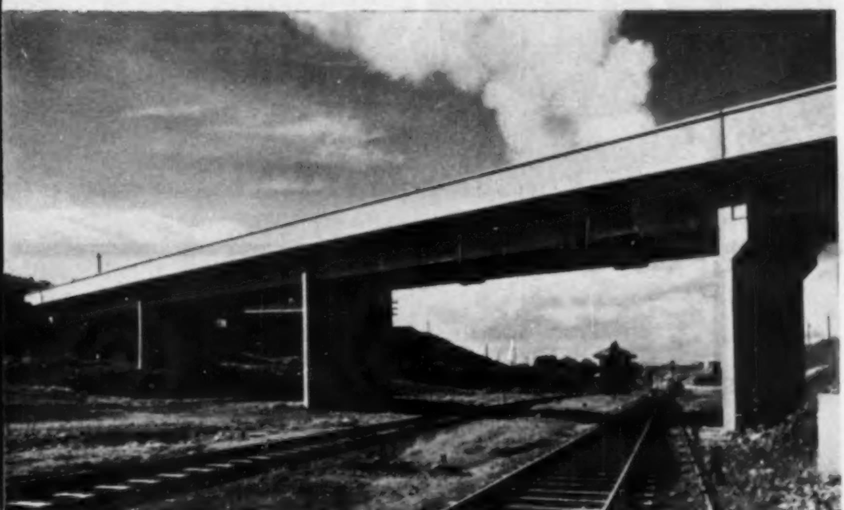

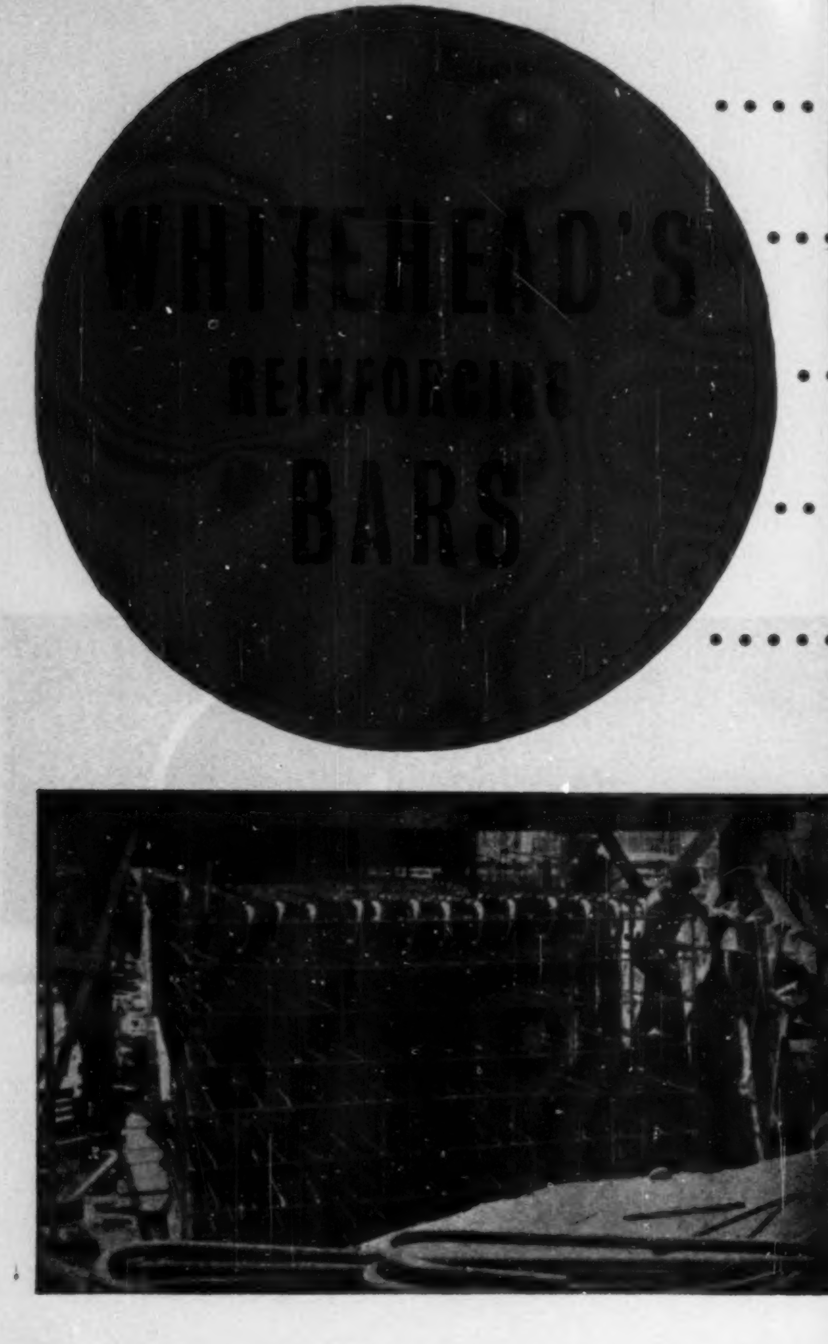
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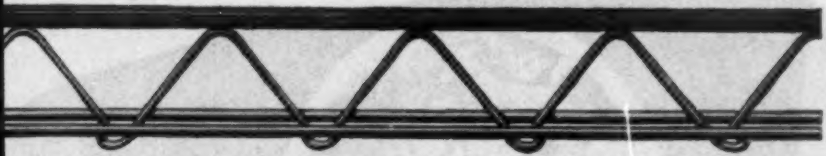

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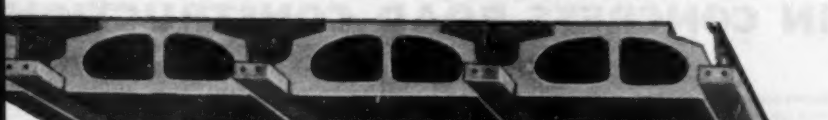

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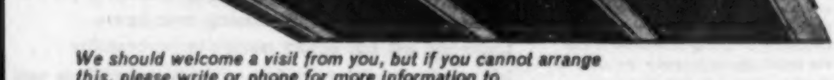

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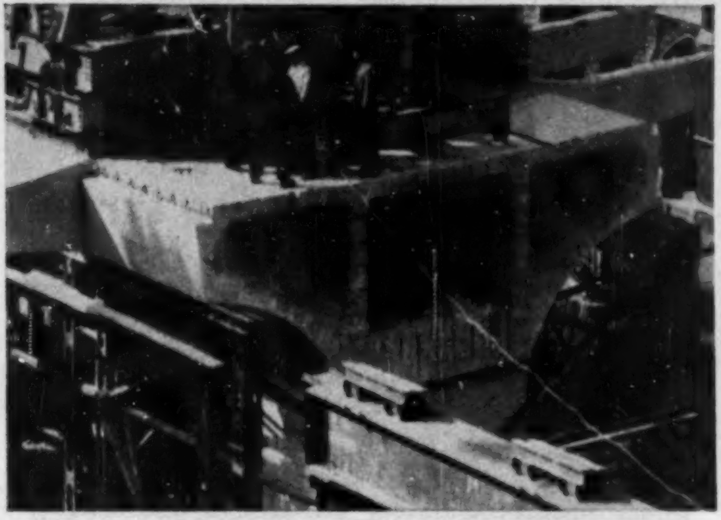

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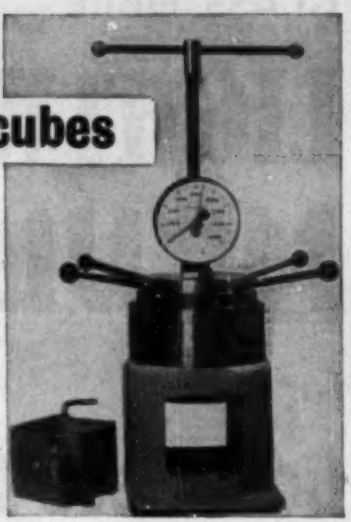

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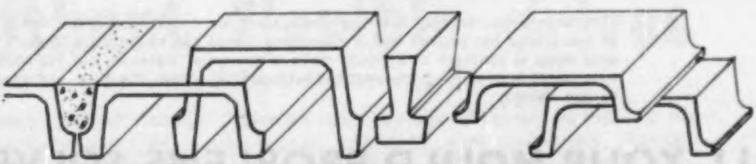

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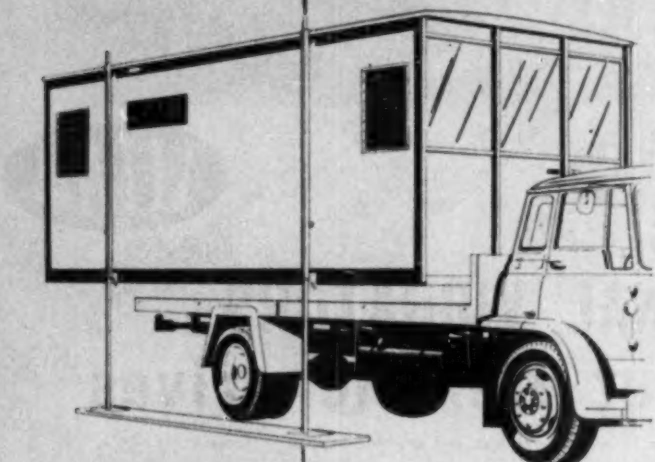

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CONCRETE AND CONSTRUCTIONAL ENGINEERING

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Volume LVI, No. 11. LONDON, NOVEMBER, 1961.

EDITORIAL NOTES

Concrete and Europe.

THE discussions taking place in political and commercial circles regarding Britain's proposed closer contact with Europe through the Common Market may not raise so many problems for the cement and concrete industries as for some other industries. Imports and exports of cement from or to the Continent do not represent a large proportion of the British cement trade. According to the statistics issued by the Organisation for European Economic Co-operation, the amount of cement exported in 1960 from the United Kingdom to countries in the European Free Trade Area was about a million tons out of a productive capacity of about fourteen million tons. In the same year the imports from this area were about 118,000 tons of cement and upwards of 300,000 tons of cement clinker. Liaison with European countries raises the question of the relative requirements of the national standards for cement. That these vary considerably, depending on the type of cement, is seen by perusal of the compilation of such standards published by Cembureau,[1] but in most respects Portland cements complying with British Standards are superior to their European counterparts. A brochure issued by a national newspaper states, in relation to the cement industry, that transport costs are a limiting factor but that the industry stands to gain if Britain enters the Common Market. On the other hand, it is stated, the reinforcement industry may " feel the pinch ".

The amount of international trade in the expanding precast concrete industry of Great Britain is, on account of the nature of the product, likely to be very limited, but the possibility of interchange of design and constructional services is no doubt being explored in some quarters. A spokesman at a recent meeting of the Reinforced Concrete Association explained that by 1963, firms in all member countries of the Common Market will be free to submit tenders for major contracts within the area, and by 1967 the more difficult problem of the free interchange of professional services may also have been solved in theory, at least, if not in practice.

What is probably of equal importance to these commercial aspects in a closer association with Europe is the possibility of there being more uniformity of design practice within the area, a condition which can hardly leave practice in this country unaffected.

An indication of some of the problems, in addition to those related to units of measurement and systems of notation, has already been seen in the activities of the European Concrete Committee which was formed in 1953 mainly for the preparation of recommendations, standardised for all European countries, to provide engineers with simple methods of calculation based on a more thorough knowledge of the behaviour of concrete and steel. During the first six years of its work the Committee endeavoured to arrive at conclusions regarding mainly the basic principles and practical methods of the calculation of the ultimate resistance of members subjected to simple bending, eccentric compression and axial compression, taking into account such factors as the rate and duration of loading, the nature of the stresses and the geometrical shape of the section. A study of the phenomenon of buckling has resulted in a method whereby the effects of a bending moment added to the normal stress are combined in the ordinary calculation of the ultimate resistance. Consideration was given to cracking of members under working load taking into account the amount of reinforcement, the mechanical and geometrical characteristics of the bars and some other conditions. Consideration of deformation, and particularly the calculations of deflections, are also the subjects of practical recommendations mainly for common types of structures. The foregoing research was concluded by considerations of the safety of structures. Other problems under investigation include resistance to shearing forces, the behaviour of ordinary slabs and flat-slabs, bond, and the phenomenon of redistribution in statically-indeterminate structures.

Progress has been made with the preparation of a code of practice for reinforced concrete construction to such an extent that draft recommendations have been issued. The present publication includes two out of three parts relating to design, but observations on the contents are best withheld until the third part, which is to include comments and appendices, is available.

In addition to the consideration of the alliance of research and design practice, there are fundamental problems which have to be solved before international uniformity of practice can be obtained. Two such problems are the conception of what constitutes failure of a reinforced concrete member and the basis on which the strength of concrete should be established. The complexity of the former problem is realised when the various current views are considered. The German and Austrian authorities define failure as occurring when arbitrary limits of the contraction of compressed concrete and elongation of the reinforcement are exceeded. Another conception, which is notably that of French engineers, defines the state of failure as occurring when the member can no longer serve its structural purpose, whether it be due to excessive cracking or excessive deformation or complete ruin of the member. Such theoretical definitions are difficult to apply since, in a test, it is not easy to verify when a particular state is reached. The measurement of the deformations at the critical section, the position of which is at first unknown, is a delicate operation. The only factor that is readily determinable is the greatest load sustained by the test specimen, but the load causing failure depends on whether the load is applied rapidly or in increments. Gradual loading with considerable periods between each increment gives more consistent results, but tests of long duration are often inconvenient and can only be carried out in especially well-equipped laboratories; interpretation of the results is, however, complicated by the occurrence of creep. These difficulties have led

the Committee to define the state of failure experimentally. The procedure recommended requires the load to be applied in specified increments throughout a period of not less than three hours but in such a manner that the test can be considered as one of short duration; the greatest load sustained is defined as the ultimate load (" charge de rupture "). The primary factors, such as deformation, compressive strain in the concrete and tensile strain in the reinforcement, are reckoned to be taken into account, and on the basis of this hypothesis a method of calculation of the ultimate resistance of members subjected to bending, direct compression, or combined stress is established. The recommendations and methods of testing are given in great detail in a report[2] published earlier this year.

The second main problem raises the question: What is the strength of concrete? As is well known, cubes, cylinders and prisms of various sizes are used in different countries as standard test pieces and the results obtained from specimens of different shapes differ considerably. Despite much research, a simple relationship between these results has not yet been established because the relationship is affected by numerous factors, the principal being the " strength " of the concrete and, to a less degree but importantly, the fineness of the aggregate. Uniformity of shape and size is aimed at by the pronouncement by the International Association of Testing Materials and Research Laboratories for Materials and Structures (RILEM), that the international standard should be a cylinder 12 in. long and 6 in. in diameter; the Committee concurs with this proposal. According to investigations, particularly those made recently in Norway, friction at the loading surfaces, which increases the apparent crushing strength of concrete, is said to be the primary cause of the varying relationship between the results obtained from cubes and cylinders. A cylinder fails where the frictional effects are least significant, and the distribution of stress is more uniform in a cylinder the height of which is at least twice its diameter. A practical advantage is that cylindrical cores can be readily cut from hardened concrete. Since failure of concrete, whether in compression or tension, is a rending apart of the constituent particles, it is probable that the tensile strength is the basic factor and, if this be so, some form of splitting test may give a truer value of the strength; in such tests it is thought that the shape of the specimen is of little importance. It may be that it is easier to correlate tensile and apparent compressive strengths than it is to correlate the crushing strengths of specimens of various shapes, and therefore some form of tensile test may yet be preferable as an international standard.

If the cylinder test is adopted throughout Europe, there will accrue the advantage that practice in Europe and the U.S.A. is alike, but some complications may be raised for such countries, Great Britain in particular, where hitherto all regulations, codes and general practice are based on the crushing strength of 6-in. cubes. The opinions of engineers in this country would be of interest, and readers are invited to contribute their views on this subject.

(1) " Cement Standards of the World." (Published by Cembureau, Sweden. Distributed in the United Kingdom by Concrete Publications Ltd. Price 25*s*.; by post 25*s*. 9*d*.)

(2) " Preparation of Practical European Standards for the Ultimate Strength Calculation of Reinforced Concrete Structures." European Committee on Concrete Report. By Y. Saillard. " Publications 1960." International Association for Bridge and Structural Engineering. (Published by Verlag Leeman, Zürich. See page 396 of this journal.)

Book Reviews.

"The Design of Cylindrical Shell Roofs." By J. E. Gibson. (London: E. & F. N. Spon Ltd. Second edition. 1961. Price 52s.)

THE revised edition of this book, the first edition of which appeared with Mr. D. W. Cooper as co-author, contains four new chapters dealing respectively with the use of electronic digital computors for the analysis of shells, reinforced cement-mortar models as a basis of the design of complex shell structures, the design of the frames supporting shell structures, and an account of the practical aspect of shell construction. The photo-elastic investigation of a rigid traverse, which was given in the previous edition, is omitted. The derivations of the formulæ used in the book are given in an appendix. The method of presentation is particularly clear and the use of one basic example throughout enables the various stresses occurring in short, intermediate and long shells to be compared. The accuracy of various approximate methods and the advantages obtained by prestressing edge beams are indicated. It is to be regretted that the opportunity was not taken, when preparing this edition, to improve the parts dealing with the design and detailing of the reinforcement in a shell, which are below the standard of the rest of the book.

"Pile Foundations." By R. D. Chellis. (London: McGraw-Hill Publishing Co., Ltd. Second edition. 1961. Price £6 4s.)

THE second edition of this book, the first of which was published ten years ago, contains much additional matter and has been brought up to date. It covers practically the entire subject of piles and piled foundations and includes a particularly good chapter on the deterioration and preservation of timber piles. The strengthening of soils by compaction, electro-osmosis, freezing and similar methods is described with a useful amount of detail. The subject matter is supported by data and numerous tables, and while some of these are also available in manufacturers' publications, the fact that so much is contained in one volume makes the new edition an essential reference book for designers and contractors dealing with foundations.

British readers may need to remember, or become familiar for the first time, with American terms which differ from those in use in this country; for example, "cushion" in place of "head packing", and "driving cap" instead of "helmet". There are, of course, differences also in practice and in the equipment normally used for particular operations, but this may make the book of more interest as it describes methods which are not well known or little used here. In many books of this type, there are particular sections which even alone would make the book a valuable source of information; the part dealing with diesel hammers is one such section.

No doubt it will not be many years before another edition is required, and when that happens more information could be usefully given on prestressed concrete piles which in recent years have come into general use in the U.S.A.

For those interested in impact formulæ, the historical background of some important formulæ is largely repeated from the earlier edition and brought up to date by references to the work of Mr. E. A. Smith. There are also useful chapters on pile-loading tests, failures of piled foundations and an appendix giving standard specifications.—D. H. L.

"Festigkeit der Biegedruckzone." By Gunter Scholz. (Berlin: Wilhelm Ernst & Son. 1960. DM. 10.70.)

THIS publication describes tests of reinforced concrete beams to determine the distribution of stress in the concrete and to establish the magnitude and position of the compressive force for heavy loads and for different qualities of concrete. The main purpose of this publication is a critical appreciation of an article entitled "The distribution of concrete stress in reinforced and prestressed concrete beams when tested to destruction by a pure bending moment", published in the "Magazine of Concrete Research", January 1951. The recommendations for relationship between stress and strain are compared with test results and the discrepancies are discussed. The conclusions, however, are so far not of practical use but are an incentive to further study.

Forces in Three-dimensional Groups of Piles.—I.

By W. E. SCRIVEN, B.Sc. (Eng.).

THE theory in this analysis is based on that proposed by Professor H. M. Westergaard[1] and is applied to three-dimensional groups of piles in general and, in particular, to some common symmetrical groups. The theory can also be applied to groups of columns if the basic assumptions are satisfied. The assumptions made are that all piles are pin-jointed at both ends, and that the pile-cap is rigid and capable of transmitting loads and displacements to the piles without itself being deformed.

General Analysis.

Consider any pile PQ (*Fig.* 1) with reference to a right-handed set of orthogonal axes OX, OY, OZ such that the co-ordinates of the junction of pile and pile-cap are x, y, z. Due to the external forces P_x, P_y, P_z and moments M_x, M_y, M_z the point x, y, z is displaced by amounts u, v, w in the directions OX, OY, OZ and is rotated through angles α, β, γ in the planes ZOY, XOZ, YOX. All movements and rotations are considered positive if they produce a shortening of the pile.

If l, m, n are the directional cosines of the pile, the total shortening due to translational movement is s_1, such that

$$s_1 = u.l + v.m + w.n. \quad . \quad . \quad . \quad . \quad (1)$$

If point C (x_0, y_0, z_0) is the centre of rotation of the group of piles, consider first

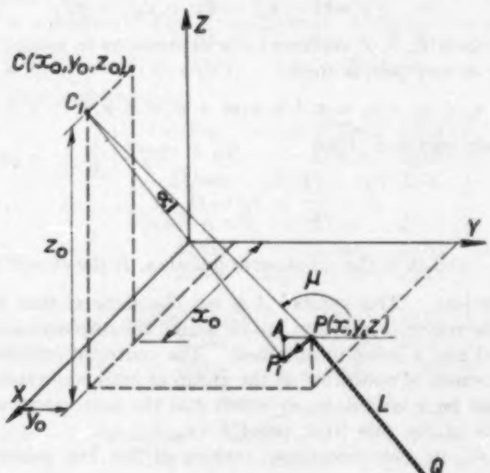

Fig. 1.

(1) H. M. WESTERGAARD. "The Resistance of a Group of Piles." Journal of Western Soc. of Eng., 1917.

the rotation in the ZOY plane (*Fig.* 1). The pile PQ is moved from P to P_1 by the rotation α. From the geometry of the figure, $PP_1 = \alpha . C_1P$;

$$A\widehat{P}P_1 = \frac{\pi}{2} - \frac{\alpha}{2} - (\pi - \mu) = \left(\mu - \frac{\alpha}{2}\right) - \frac{\pi}{2};$$

$$\cos A\widehat{P}P_1 = \sin\left(\mu - \frac{\alpha}{2}\right) = \sin\mu \text{ nearly; and}$$

$$\sin A\widehat{P}P_1 = \cos\left(\mu - \frac{\alpha}{2}\right) = \cos\mu \text{ nearly.}$$

But $$\cos\mu = -\frac{(y-y_0)}{C_1P} \quad \text{and} \quad \sin\mu = \frac{z_0 - z}{C_1P}; \text{ hence}$$

$$AP = PP_1 \cos A\widehat{PP_1} = -\alpha(z-z_0) \quad \text{and} \quad AP_1 = PP_1 \sin \widehat{A}PP_1 = -\alpha(y-y_0).$$

The co-ordinates of P_1 are therefore $[x,\ y - \alpha(z - z_0),\ z - \alpha(y - y_0)]$, and the co-ordinates of the remote end Q of the pile are $[x + Ll,\ y + Lm,\ z - Ln]$, in which L is the length of the pile. Hence the shortening of the pile due to the rotation α is

$$\begin{aligned} s_\alpha &= L - \{[x+Ll-x]^2 + [y+Lm-y+\alpha(z-z_0)]^2 + [z-Ln-z+\alpha(y-y_0)]^2\}^{\frac{1}{2}} \\ &= L - \{L^2 + 2\alpha L[m(z-z_0) - n(y-y_0)]\}^{\frac{1}{2}}, \text{ ignoring terms in } \alpha^2. \end{aligned}$$

Expanding by the binomial theorem and ignoring second and subsequent terms,

$$s_\alpha = \alpha[n(y - y_0) - m(z - z_0)] = \alpha R,$$

and similarly $$s_\beta = \beta[l(z - z_0) - n(x - x_0)] = \beta S,$$

and $$s_\gamma = \gamma[m(x - x_0) - l(y - y_0)] = \gamma T,$$

in which the symbols R, S, T represent the expressions in square brackets. The total shortening in any pile is then

$$s_T = s_1 + s_\alpha + s_\beta + s_\gamma = u.l + v.m + w.n + \alpha.R + \beta.S + \gamma.T \quad . \quad (2)$$

If the force in any pile is F, then

$$F = \frac{AE}{L} \times s_T = Q.s_T \quad . \quad . \quad . \quad . \quad . \quad (3)$$

in which $Q = \dfrac{AE}{L}$ and A is the cross-sectional area, E the elastic modulus, and L the length of the pile. [This symbol A is not the same as that in equation (4).]

Clearly, if the centre of rotation can be found, the displacements and rotations can be calculated and a solution obtained. The centre of rotation must be such that the least moment of resistance of the group of piles is obtained at that point. Hence ΣQp^2 must be a minimum, in which p is the least perpendicular distance of the centre-line of the pile from point C (x_0, y_0, z_0).

If L', M', N' are the directional cosines of the line joining P (x, y, z) to C (x_0, y_0, z_0) (*Fig.* 2), then

$$L', M', N' = \frac{(x - x_0), (y - y_0), (z - z_0)}{\sqrt{(x - x_0)^2 + (y - y_0)^2 + (z - z_0)^2}}$$

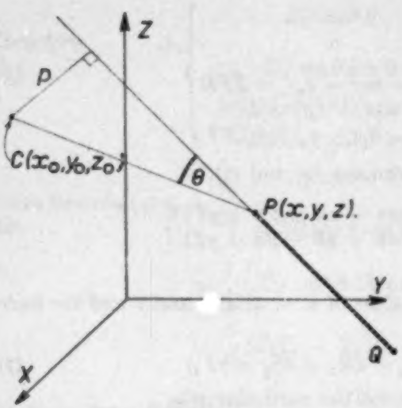

Fig. 2.

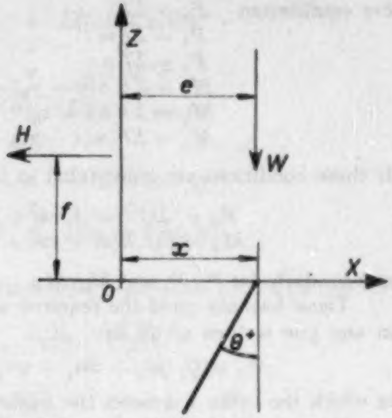

Fig. 3.

and if θ is the angle between the centre-line of the pile and the line PC,

$$\begin{aligned} p &= \mathrm{PC}\sin\theta \\ &= \left[\sqrt{(x-x_0)^2+(y-y_0)^2+(z-z_0)^2}\right]\left[\sqrt{\Sigma(Mn-Nm)^2}\right] \\ &= \sqrt{\Sigma[n(y-y_0)-m(z-z_0)]^2} \\ &= [R^2+S^2+T^2]^{\frac{1}{2}}, \end{aligned}$$

in which Σ denotes the sum of the three cyclic terms.

Now since ΣQp^2 is a minimum $\Sigma Qp\dfrac{\partial p}{\partial x_0} = \Sigma Qp\dfrac{\partial p}{\partial y_0} = \Sigma Qp\dfrac{\partial p}{\partial z_0} = 0.$

Hence if $p \neq 0$, $\Sigma Q(nS - mT) = \Sigma Q(lT - nR) = \Sigma Q(mR - lS) = 0.$

The solutions of these equations are

$$\left.\begin{aligned} x_0 &= +\frac{1}{\Delta_1}\begin{vmatrix} -\Sigma Qlm & -\Sigma Qln & \Sigma QA \\ \Sigma Q(l^2+n^2) & -\Sigma Qmn & \Sigma QB \\ -\Sigma Qmn & \Sigma Q(l^2+m^2) & \Sigma QC \end{vmatrix} \\ y_0 &= -\frac{1}{\Delta_1}\begin{vmatrix} \Sigma Q(m^2+n^2) & -\Sigma Qln & \Sigma QA \\ -\Sigma Qlm & -\Sigma Qmn & \Sigma QB \\ -\Sigma Qln & \Sigma Q(l^2+m^2) & \Sigma QC \end{vmatrix} \\ z_0 &= +\frac{1}{\Delta_1}\begin{vmatrix} \Sigma Q(m^2+n^2) & -\Sigma Qlm & \Sigma QA \\ -\Sigma Qlm & \Sigma Q(l^2+n^2) & \Sigma QB \\ -\Sigma Qln & -\Sigma Qmn & \Sigma QC \end{vmatrix} \end{aligned}\right\} \quad . \quad . \quad (4)$$

in which $\Delta_1 = -\begin{vmatrix} \Sigma Q(m^2+n^2) & -\Sigma Qlm & -\Sigma Qln \\ -\Sigma Qlm & \Sigma Q(l^2+n^2) & -\Sigma Qmn \\ -\Sigma Qln & -\Sigma Qmn & \Sigma Q(l^2+m^2) \end{vmatrix}$

and

$$\begin{aligned} A &= -(m^2+n^2)x + lmy + lnz; \\ B &= lmx - (l^2+n^2)y + mnz; \\ C &= lnx + mny - (l^2+m^2)z. \end{aligned}$$

For equilibrium
$$\left.\begin{aligned} P_x &= \Sigma Fl \\ P_y &= \Sigma Fm \\ P_z &= \Sigma Fn \\ M_x &= \Sigma F[n(y-y_0) - m(z-z_0)] = \Sigma FR \\ M_y &= \Sigma F[l(z-z_0) - n(x-x_0)] = \Sigma FS \\ M_z &= \Sigma F[m(x-x_0) - l(y-y_0)] = \Sigma FT \end{aligned}\right\} \quad . \quad . \quad . \quad (5)$$

If these conditions are substituted in formulæ (2) and (3),

$$\left.\begin{aligned} P_x &= \Sigma Q.l(ul + vm + wn + \alpha R + \beta S + \gamma T) \\ M_x &= \Sigma Q.R(ul + vm + wn + \alpha R + \beta S + \gamma T) \end{aligned}\right\} \quad . \quad . \quad (6)$$

and similarly for P_y, P_z and M_y, M_z.

These formulæ yield the required values of u, v, w, α, β and γ and the force in any pile is then given by

$$F_1 = Q_1\,(ul_1 + vm_1 + wn_1 + \alpha R_1 + \beta S_1 + \gamma T_1) \quad . \quad . \quad (7)$$

in which the suffix 1 denotes the values for the particular pile.

MOMENT NOTATION.—External non-axial loads on the pile group cause moments, the magnitude and direction of which can be expressed in terms of the last three of formulæ (5). Suppose, for instance, that in any group a force P_y acts at a distance z from, and parallel to, the OY axis, and a force P_z acts at a distance y from, and parallel to, the OZ axis; the applied moment, from (5), is then $M_x = P_z(y - y_0) - P_y(z - z_0)$. The sign convention is in accordance with the selected set of axes, a positive value of M_x tending to rotate the Y-axis clockwise about the X-axis into the Z-axis (*Fig.* 4).

REDUCTION TO TWO-DIMENSIONAL SYSTEM.—Before considering particular three-dimensional cases it is of interest to investigate the reduction of the general solution to a two-dimensional case where all pile-heads are at the same level. The notation is shown in *Fig.* 3, and the following abbreviations are used.

$$\Sigma_1 = \Sigma Q \cos^2\theta;\ \Sigma_2 = \Sigma Q \sin\theta\cos\theta;\ \Sigma_3 = \Sigma Qx\cos^2\theta;$$
$$\Sigma_4 = \Sigma Qx \sin\theta\cos\theta;\ \Sigma_5 = \Sigma Q\sin^2\theta;\ \Sigma_6 = \Sigma QX^2\cos^2\theta,$$

in which $X = (x - x_0) + z_0 \tan\theta$.

Now $\quad l = \cos(\pi/2 + \theta) = -\sin\theta, \quad m = 0, \quad$ and $\quad n = -\cos\theta;$

$$P_x = -H, \quad P_y = 0, \quad P_z = -W,$$

and $\quad M_y = P_x(f - z_0) - P_z(e - x_0) = W(e - x_0) + H(z_0 - f).$

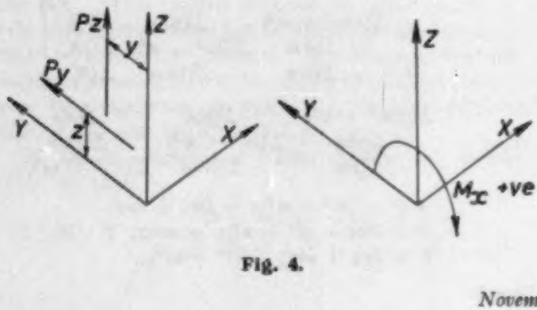

Fig. 4.

Therefore $$\Delta_1 = -\begin{vmatrix} \Sigma Q\cos^2\theta & 0 & \Sigma Q\sin\theta\cos\theta \\ 0 & 1 & 0 \\ \Sigma Q\sin\theta\cos\theta & 0 & \Sigma Q\sin^2\theta \end{vmatrix}$$
$$= -\Sigma Q\cos^2\theta\Sigma Q\sin^2\theta + (\Sigma Q\sin\theta\cos\theta)^2$$
$$= -\Sigma_1\Sigma_5 + (\Sigma_2)^2 = -G.$$

From formula (4) $$z_0 = \frac{1}{\Delta_1}\begin{vmatrix} \Sigma Q\cos^2\theta & 0 & -\Sigma Qx\cos^2\theta \\ 0 & 1 & 0 \\ -\Sigma Q\sin\theta\cos\theta & 0 & \Sigma Qx\sin\theta\cos\theta \end{vmatrix}$$
$$= \frac{1}{\Delta_1}\left\{\Sigma Q\cos^2\theta\,\Sigma Qx\sin\theta\cos\theta - \Sigma Q\sin\theta\cos\theta\,\Sigma Qx\cos^2\theta\right\}$$
$$= \frac{1}{\Delta_1}(\Sigma_1\Sigma_4 - \Sigma_2\Sigma_3) = \frac{\Sigma_2\Sigma_3 - \Sigma_1\Sigma_4}{G}.$$

Similarly from formula (4), $x_0 = \dfrac{\Sigma_3\Sigma_5 - \Sigma_2\Sigma_4}{G}$.

Also $R = 0$, $S = z_0\sin\theta + (x - x_0)\cos\theta = X\cos\theta$, and $T = 0$; formulæ (6) can therefore be written in the form
$$\begin{aligned} u\Sigma Q\sin^2\theta + w\Sigma Q\sin\theta\cos\theta - \beta\Sigma QX.\sin\theta\cos\theta + H &= 0 \\ u\Sigma Q\sin\theta\cos\theta + w\Sigma Q\cos^2\theta - \beta\Sigma QX.\cos^2\theta + W &= 0 \\ -u\Sigma QX.\sin\theta\cos\theta + w\Sigma QX.\cos^2\theta + \beta\Sigma QX^2.\cos^2\theta - M &= 0. \end{aligned}$$
But $X\sin\theta\cos\theta = z_0\sin^2\theta + (x - x_0)\sin\theta\cos\theta = z_0\Sigma_5 + \Sigma_4 - x_0\Sigma_2 = 0$. Similarly $\Sigma QX\cos^2\theta = z_0\Sigma_2 + \Sigma_3 - x_0\Sigma_1 = 0$ and the foregoing formulæ reduce to
$$\begin{aligned} u\Sigma_5 + w\Sigma_2 + H &= 0 \\ u\Sigma_2 + w\Sigma_1 + W &= 0 \\ \beta\Sigma_6 - M &= 0. \end{aligned}$$

From these, $u = \dfrac{W\Sigma_2 - H\Sigma_1}{-G}$; $w = \dfrac{W\Sigma_5 - H\Sigma_2}{-G}$; and $\beta = \dfrac{M}{\Sigma_6}$, and, substituting in formula (7),
$$F_1 = \{u\sin\theta_1 - w\cos\theta_1 + \beta[z_0\sin\theta_1 + (x_1 - x_0)\cos\theta_1]\}Q_1$$
$$= \cos\theta_1\left\{\frac{H\Sigma_1 - W\Sigma_2}{G}\tan\theta_1 - \frac{H\Sigma_2 - W\Sigma_5}{G} + \frac{M.X_1}{\Sigma_6}\right\}Q_1$$
$$= \cos\theta_1\left\{W\left[\frac{\Sigma_5 - \Sigma_2\tan\theta_1}{G}\right] + H\left[\frac{\Sigma_1\tan\theta_1 - \Sigma_2}{G}\right] + \frac{M.X_1}{\Sigma_6}\right\}Q_1.$$
From this result the force in any pile may be determined.

Circular Groups.

Consider the case in which the piles are grouped symmetrically about a circle of radius a in the plane XOY; that is, for all piles $z = 0$. The co-ordinate system is illustrated in *Fig.* 5, from which
$$x = a\sin\theta; \quad y = a\cos\theta \quad . \quad . \quad . \quad . \qquad (8)$$

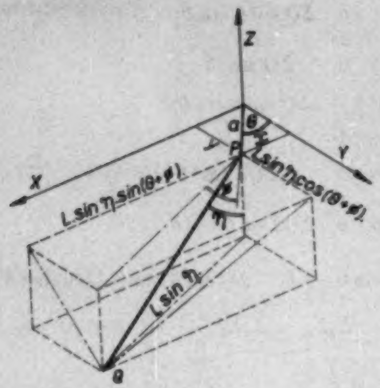

Fig. 5.

NOTE.—In the analysis on pages 376 to 378, the symbol μ represents angle η in this diagram.

The directional cosines of PQ are then

$$l = \sin\mu . \sin(\theta + \phi); \qquad m = \sin\mu . \cos(\theta + \phi); \qquad n = -\cos\mu \quad . \quad (9)$$

If all piles have equal values of Q, then by symmetry

$$\Sigma Qmn = \Sigma Qln = \Sigma Qlm = \Sigma Qmnx = \Sigma Qlny = \Sigma Qlmx = \Sigma Ql^2x$$
$$= \Sigma Ql^2y = \Sigma Qlmy = \Sigma Qm^2x = \Sigma Qm^2y = \Sigma Qn^2x = \Sigma Qn^2y = 0.$$

Hence $\qquad A = B = 0 \quad \text{and} \quad C = \Sigma Qlnx + \Sigma Qmny.$

Also $\qquad \Delta_1 = -\Sigma Q(l^2 + m^2).\Sigma Q(m^2 + n^2).\Sigma Q(l^2 + n^2),$

$$x_0 = y_0 = 0, \quad \text{and} \quad z_0 = \frac{1}{\Delta_1}\{\Sigma Q(m^2 + n^2).\Sigma Q(l^2 + n^2).\Sigma QC\}$$

$$= \frac{1}{\Sigma Q(l^2 + m^2)}(\Sigma Qlnx + \Sigma Qmny).$$

Substituting from equation (9),

$$z_0 = \frac{1}{\sin^2 y\Sigma Q} \times a \ \sin\mu \ \cos\mu\{\Sigma Q \sin(\theta + \phi)\sin\theta + \Sigma Q\cos(\theta + \phi)\cos\theta\}$$

$$= \frac{a\sin\mu\cos\mu\cos\phi}{\sin^2\mu} = a\cot\mu\cos\phi \quad . \quad . \quad . \quad . \quad . \quad . \quad (10)$$

Substituting in equation (5),

$$R = n.y + m.z_0; \quad S = -l.z_0 - n.x; \quad T = m.x - l.y,$$

and since $\Sigma Qn^2x.y = \Sigma Qmnxy = \Sigma Qlny^2 = 0$, and

$$\Sigma QR^2 = \Sigma Qn^2y^2 + z_0^2\Sigma Q.m^2 + 2z_0\Sigma Qmny;$$
$$\Sigma QS^2 = z_0^2\Sigma Ql^2 + \Sigma Qn^2x^2 + 2z_0\Sigma Qln.x;$$
$$\Sigma QT^2 = \Sigma Qm^2x^2 + \Sigma Ql^2y^2; \quad \text{it follows that}$$
$$\Sigma QmR = \Sigma Qmny + z_0\Sigma Qm^2; \quad \Sigma QlS = -z_0\Sigma Ql^2 - \Sigma Qlnx;$$
$$\Sigma QlR = \Sigma QnR = \Sigma QmS = \Sigma QnS = \Sigma QlT = \Sigma QRS = \Sigma QmT$$
$$= \Sigma QTR = \Sigma QnT = \Sigma QTS = 0.$$

Assume now that all piles have the same angles μ and ϕ. Let there be N piles,

spaced at angular intervals of δ; then $\delta = \frac{2\pi}{N}$. The summations required for the solution are

$$\Sigma Ql^2 = \sin^2\mu\Sigma Q\sin^2(\theta+\phi); \quad \Sigma Qm^2 = \sin^2\mu.\Sigma Q\cos^2(\theta+\phi);$$
$$\Sigma Qn^2 = \cos^2\mu\Sigma Q; \quad \Sigma Qlnx = -a\sin\mu\cos\mu.\Sigma Q\sin(\theta+\phi)\sin\theta;$$
$$\Sigma Qmny = -a\sin\mu\cos\mu.\Sigma\cos(\theta+\phi)\cos\theta;$$
$$\Sigma Ql^2x^2 = a^2\sin^2\mu.\Sigma Q\sin^2(\theta+\phi)\sin^2\theta;$$
$$\Sigma Qm^2x^2 = a^2\sin^2\mu.\Sigma Q\cos^2(\theta+\phi)\sin^2\theta;$$
$$\Sigma Qn^2x^2 = a^2\cos^2\mu.\Sigma Q\sin^2\theta; \quad \Sigma Ql^2y^2 = a^2\sin^2\mu.\Sigma Q\sin^2(\theta+\phi)\cos^2\theta;$$
$$\Sigma Qn^2y^2 = a^2\cos^2\mu.\Sigma Q\cos^2\theta.$$

It can be shown that

$$\sin\theta_1 + \sin(\theta_1+\delta) + \sin(\theta_1+2\delta) + \ldots + \sin(\theta_1+[N-1]\delta) = \frac{\sin\left[\theta_1 + \left(\frac{N-1}{2}\right)\delta\right]\sin\frac{N\delta}{2}}{\sin\frac{\delta}{2}}$$

and

$$\cos\theta_1 + \cos(\theta_1+\delta) + \cos(\theta_1+2\delta) + \ldots + \cos(\theta_1+[N-1]\delta) = \frac{\cos\left[\theta_1 + \left(\frac{N-1}{2}\right)\delta\right]\sin\frac{N\delta}{2}}{\sin\frac{\delta}{2}}.$$

Let $\theta_1 = \frac{\delta}{2} = \frac{\pi}{N}$; hence $\Sigma\sin\theta = \sin^2\pi\operatorname{cosec}\frac{\pi}{N} = \Sigma\cos\theta$

$$= \tfrac{1}{2}\sin 2\pi\operatorname{cosec}\frac{\pi}{N} = 0.$$

Similarly $\Sigma\sin 2\theta = \Sigma\cos 2\theta = \Sigma\sin 4\theta = \Sigma\cos 4\theta = 0.$

Substituting these identities in the summations assuming Q to be constant,

$$\Sigma Ql^2 = \Sigma Qm^2 = \frac{N}{2}\sin^2\mu; \quad \Sigma Qnlx = \Sigma Qmny = -a\frac{N}{2}\sin\mu\cos\mu\cos\phi;$$
$$\Sigma Qn^2 = N\cos^2\mu; \quad \Sigma Ql^2x^2 = \frac{a^2N}{4}\sin^2\mu(1+2\cos\phi);$$
$$\Sigma Qm^2x^2 = \Sigma Ql^2y^2 = \frac{a^2N}{4}\sin^2\mu(1-2\cos\phi);$$
$$\Sigma Qn^2x^2 = \Sigma Qn^2y^2 = \frac{a^2N}{2}\cos^2\mu; \quad \Sigma QmR = \Sigma QlS = 0;$$
$$\Sigma QR^2 = \Sigma QS^2 = \frac{a^2N}{2}\cos^2\mu\sin^2\phi; \quad \Sigma QT^2 = \frac{a^2N}{2}\sin^2\mu(1-2\cos\phi).$$

Substituting these results into equation (6),

$$\left.\begin{aligned} u = \frac{2P_x}{N\sin^2\mu}; \quad v = \frac{2P_y}{N\sin^2\mu}; \quad w = \frac{P_z}{N\cos^2\mu}; \quad \alpha = \frac{2M_x}{a^2N\cos^2\mu\sin^2\phi}; \\ \beta = \frac{2My}{a^2N\cos^2\mu\sin^2\phi}; \quad \gamma = \frac{2M_z}{a^2N\sin^2\mu(1-2\cos\phi)}. \end{aligned}\right\} \quad (11)$$

The force in any pile is found by substitution in equation (7), which can then be reduced to give

$$F_\theta = \frac{2P_x.\sin(\theta+\phi)}{N\sin\mu} + \frac{2P_y.\cos(\theta+\phi)}{N\sin\mu} - \frac{P_z}{N\cos\mu} - \frac{2M_x.\sin(\theta+\phi)}{aN\cos\mu\sin\phi}$$
$$- \frac{2M_y.\cos(\theta+\phi)}{aN\cos\mu\sin\phi} + \frac{2M_z.\cos\phi}{aN\sin\mu(1-2\cos\phi)} \quad . \quad (12)$$

Equation (12) enables any group of piles to be solved if all the piles have equal values of a, μ and ϕ, and there is only one pile at each node point. The problem of several piles at each node, with different values of μ and ϕ, is now considered, but in order to reduce the amount of algebra involved only the moments and forces acting in one plane (ZOY) are considered. For this case $P_x = M_y = M_z = 0$, and

$$F_\theta = P_y\frac{2\cos(\theta+\phi)}{N\sin\mu} - P_z\frac{1}{N\cos\mu} - M_x\frac{2\sin(\theta+\phi)}{aN\cos\mu\sin\phi} \quad . \quad . \quad (13)$$

If the values of a, μ and ϕ are different for each group, then

$$v = \frac{2P_y}{\Sigma N\sin^2\mu}; \quad w = \frac{P_z}{\Sigma N\cos^2\mu}; \quad \alpha = \frac{2M_x}{\Sigma a^2 N\cos^2\mu\sin^2\phi};$$

and

$$z_0 = \frac{\Sigma aN\sin\mu\cos\mu\cos\phi}{\Sigma N\sin^2\mu} \quad . \quad (14)$$

Hence

$$F^\theta_{a,\mu,\phi_1} = v\{\sin\mu_1\cos(\theta+\phi_1)\} - w\cos\mu_1 - \alpha\{a_1\sin(\theta+\phi_1)\sin\phi_1\cos\mu_1\} \quad (15)$$

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FIG. 1.

A Bridge in Northern Ireland.

By JOHN FABER, B.Sc., M.I.C.E., M.I.Struct.E.

A REINFORCED concrete bridge (*Fig.* 1) 320 ft. long was recently constructed, for the British Portland Cement Manufacturers Ltd., near the quarry serving the cement works at Magheramorne, near Larne, Northern Ireland. To win chalk for the manufacture of cement, it is necessary to dispose of a basalt overburden at the rate of 2,000,000 tons per year. The basalt is being tipped into Larne Lough with a view that eventually a causeway may be formed across the Lough to Island Magee. From the quarry on the west side, the new concrete bridge gives access directly over a public main road and the Belfast–Larne railway to Larne Lough on the east side.

Design Requirements.

For the past fifteen years, the road and railway were crossed by dumpers of medium size using a temporary steel bridge, so that a considerable tip of basalt has already been formed in the Lough. Therefore the eastern end of the new bridge had to be founded on the tip. Another requirement was that the line of the bridge should be arranged to suit a possible future belt-conveyor to be used as an alternative method of transporting the basalt. These requirements, together with the disposition of the three railway tracks and the improved line for the main road, led to siting the bridge on a skew and providing three main spans each 81 ft. long.

The large dumpers, which the bridge is designed to carry, are 32½ ft. long and 12½ ft. wide, and weigh 70 tons when fully laden. In case a dumper breaks down on the bridge, another dumper would go on the bridge to push it off and thus two machines can be on a span at the same time. Other loads and forces have been provided for as follows. Due to the surface of the carriageway being uneven because of spillings from the vehicles, a vertical impact effect of 50 per cent. of the weight of one laden vehicle is assumed. Due to the very efficient braking, a longitudinal thrust equal to the laden weight of the vehicle is taken into account. Due to lateral sway, the wheels on one side are assumed to carry five-eighths of the total load of the vehicle.

The requirement that boulders spilled from the dumpers should be prevented from falling on to the railway or road has led to the adoption of resilient walkways on both sides of the bridge, with parapets of substantial construction though not of unusual height. These measures give the bridge a sober æsthetic appearance, whereas higher parapets on a narrower deck would have looked curious and would have cost more.

Alternative Designs of the Deck.

Alternative designs of the deck were considered (*Fig.* 3). The first scheme was to provide precast prestressed trough-shaped beams with post-tensioned steel, there being four beams side by side on each span. The beams were to have been precast on the existing tip in the Lough, and launched by suspension from a steel lattice girder so that each beam would be suspended at two points near its ends. When the beams had been placed for the easternmost span, the deck slab for that span would be cast insitu, and would act as the compression flange when the bridge was in service. The precast beams for the next span would then be launched over the first span, and so on.

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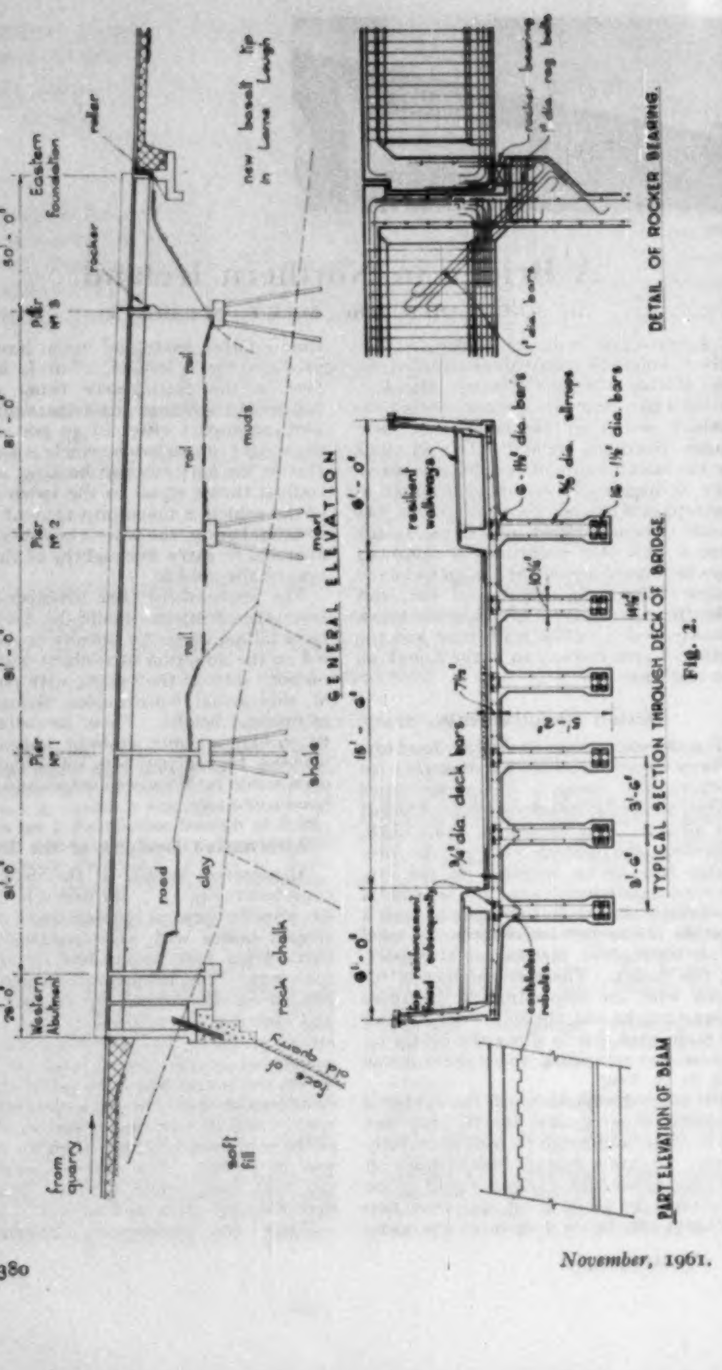

GENERAL ELEVATION
from quarry
soft fill
face of old quarry
Western Abutment
rock chalk
clay
road
Pier No 1
shale
marl
Pier No 2
muds
Pier No 3
rocker
Eastern foundation
roller
new basalt tip in Larne Lough
26'-0"
81'-0"
81'-0"
81'-0"
50'-0"
TYPICAL SECTION THROUGH DECK OF BRIDGE
8'-0"
15'-6"
8'-0"
resilient walkways
⅝" dia deck bars
castellated rebates
6 - 1¼" dia. bars
⅜" dia. stirrups
18 - 1¼" dia. bars
3'-6"
3'-6"
DETAIL OF ROCKER BEARING.
1" dia bars
rocker bearing
PART ELEVATION OF BEAM

Fig. 2.

being prepared, the ground investigations showed that the so-called basalt tip was so interspersed with clay filling, which was insufficiently consolidated, that no horizontal thrust could be resisted at the eastern end of the bridge; also the provision of adequate vertical support would most likely be difficult. The western abutment, however, would be on hard chalk, and it was decided therefore to anchor all the horizontal thrust back to this end. Precasting and launching from the western end would not have been convenient due to restrictions of the site. Also, the provision of an anchorage at one end only would mean building the bridge in one length without intermediate expansion joints; the great variation in length this method would cause under the effects of temperature, creep, shrinking and braking forces did not suit the scheme with prestressed precast members. This scheme was therefore abandoned.

A second design was therefore developed, and included six reinforced concrete beam-ribs cast insitu on a continuous bird-cage scaffolding. By providing slightly longer piles under the intermediate piers, and by inclining the piles at 1 in 10, it was thought that the settlement would be small enough for the bridge to be designed as a fully continuous structure. (In the precast scheme the beams would have acted as freely-supported members, at least under the influence of all dead loads.) The cost of the scaffolding was found to be less than the combined cost of launching the precast beams combined with the staging subsequently required for the deck slab. The merit of full continuity was that there was an overall saving in cost of the cast-insitu bridge compared with the precast scheme. Furthermore, the reinforced concrete structure as adopted is probably better suited than the prestressed concrete scheme to resist the heavy impact forces.

The Beams.

The dimensions of the beam-ribs were considered in conjunction with the contractor with a view to repetitive use of the shuttering, and to give sufficient space between the ribs for ease of subsequent removal (*Fig.* 2). The normal objection to a cast-insitu bridge is the cost of providing the support for one and a half spans simultaneously necessary to avoid the development of deflections and bending moments contrary to the intentions of a fully continuous design; but, by the use of a highly repetitive system of shuttering on the continuous bird-cage scaffolding, this objection does not apply.

The spacing of the beam-ribs and location of transverse diaphragms were such that wherever a vehicle might be on the width of the deck, its weight would be shared by at least five of the six ribs provided. The dimensions of the ribs and the spacing of the reinforcement were arranged to facilitate the use of immersion vibrators for the whole of the work. Prior to construction, the contractor made up a dummy length of beam-rib, 5 ft. long, to determine the effect of the vibrators in the actual cross-section with concrete containing angular crushed basalt which is the only aggregate available in the district.

Foundations.

Ground investigation showed that the west abutment would be on a sloping face of hard chalk. However, it was not discovered until later that part of this chalk had been quarried to a depth of at least 50 ft. (and probably more), and this area of the quarry working had been filled in subsequently with softer material. This necessitated building up the abutment from new ledges cut into the quarry face, a feature which caused some delay but which greatly enhanced the effectiveness of the abutment as an anchorage. The projecting keys at the lower part of the abutment were reinforced with rails which projected into the walls of the hollow-box structure forming the abutment (*Fig.* 2).

The intermediate piers are founded on a variety of faulted strata, and piled foundations are provided. The piles are of the bored type owing to the inaccessibility of the site and the close proximity of the railway tracks. Clearly a filling of mud had been deposited at the time the railway was constructed, and in boring for the piles it was wished that the engineers who had placed this filling had first removed the wrecks of timber ships lying on the old shore of the Lough. A number of piles were concreted with the boreholes partly filled with water, but two of these piles were tested and proved satisfactory. In service the piled foundations have not moved a measurable amount. At the easternmost pier it was expected

that the basalt tip in the Lough would have squeezed downwards and sideways into the mud, and subsequently this was found to be so. This necessitated excavating to a depth of about 6 ft. immediately adjacent to the railway line before boring for the piles could be started.

At the extreme eastern end of the bridge, where the end span bears on the basalt tip, the design of the foundation was necessarily uncertain. The boulders of basalt made it impracticable to bore or drive piles, and there was no easy means of obtaining samples to determine the strength and degree of consolidation of the clay. Accordingly, excavation for a spread foundation was commenced, and the size and level of the base were determined from observations made as the work proceeded. Eventually it was decided to bear on the clay filling at a pressure of $\frac{1}{2}$ ton per square foot. Roller bearings were provided at this support to ensure concentric loading and avoidance of lateral thrust. Before constructing the foundation, the ground was subjected to severe pounding from a 3-ton piling-hammer, and this treatment produced a preliminary consolidation of about 1 ft. In anticipation of further settlement, the formation was constructed 2 in. high. A settlement of $\frac{1}{2}$ in had taken place by the time the bridge had been in service for two months, but the total settlement after eight months' service was still only $\frac{5}{8}$ in.

In view of the uncertainties as to what foundations could actually be provided at reasonable cost, the design of the superstructure was not finally decided until after the construction of the various foundations was well advanced.

Superstructure.

The design of the superstructure of the bridge is shown in *Fig.* 2. The anchor of the whole system is the western box-abutment. The three 81-ft. spans, in conjunction with the western abutment, form a rigid unit. This facilitated the constructional work, which proceeded eastwards from the western abutment. The provision of hinges at the tops and bottoms of the intermediate piers was considered, but this was found to be more expensive than providing rigid joints, despite the horizontal thrusts imparted to the piers by such effects as temperature, shrinking, and braking forces. The piers are deliberately thin (18 in.), and are each

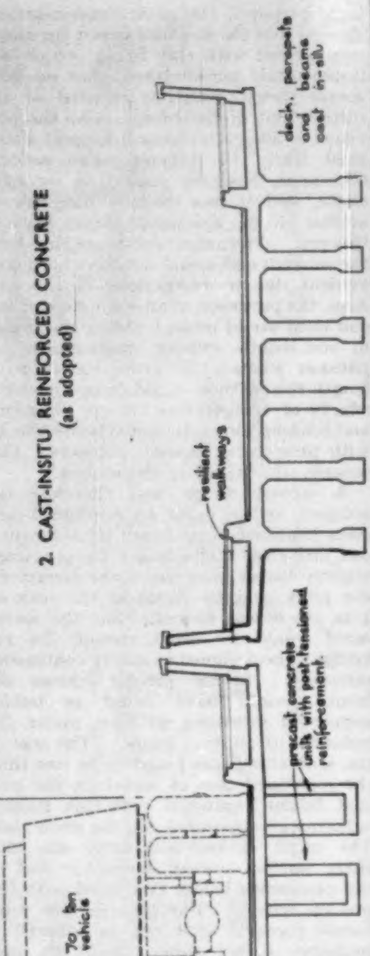


Fig. 3.

reinforced differently owing to the different lateral displacements to which they are each subjected. At the eastern face of the Pier No. 3, a pin connection is provided for the short span across to the basalt tip, and this allows for settlement of the foundation on the tip itself. However, the 70-ton braking force from the end span has to be transferred in tension to the western abutment, and this is achieved by the detail indicated at *Fig.* 2. As a secondary precaution, the end span is prevented from riding off its bearings by reinforced concrete claws which grip under the end stiffening diaphragm of the deck.

Constructional Details.

To assist the proper compaction of the concrete in the beam-ribs, they were cast up to a level 7½ in. below the soffit of the deck slab, and there provided with castellated rebates as shown at *Fig.* 2. The continuity reinforcement in the top of the deck was fixed subsequently and incorporated in the slab when this was concreted as a separate operation. The castellations have to transfer, between the beam-ribs and the slab, the normal horizontal shearing forces which arise from the bending actions of the bridge deck. However, care had to be taken to prevent shrinking being restricted by the castellations with consequent transverse cracking of the slab, or alternatively the breaking off of the castellations. For this reason the slab was cast in lengths of 40 ft. and precautions were taken to minimise the shrinkage.

The provision of a complete bird-cage scaffolding enabled each span to be in full continuity with its neighbours before the soffit shutters were removed. It also facilitated speedy working, and in fact different stages of the construction on each of the spans were being operated simultaneously.

At one stage there was a report that blasting in the quarry "lifted a man inches off the ground", and it was wondered whether this would be detrimental to the concrete in the earlier stages of hardening. Accordingly a careful watch was kept, and as a result it indicated that the effect of blasting was more psychological than physical, and the man who was lifted off the ground must have been either highly nervous or very agile. However, as a precaution, the undersides of the parapet copings were recessed ½ in. to provide a key with the insitu walling.

The new approach road on the basalt tip is of flexible construction formed as follows. The tip had to be built up, and advantage was taken of this operation by first laying a basalt sub-base 2 ft. thick, on which was deposited a rolled dry-lean concrete base 12 in. thick, which in turn was covered by an asphalt surfacing 2½ in. thick. The sub-base was laid in 9-in. layers and consolidated by an 8-ton roller. The dry-lean concrete base was laid in three 4-in. layers, each of which was consolidated by a vibrating roller weighing 30 cwt. The mixture for the dry-lean concrete is 1 : 22 by weight. The flexible paving has proved entirely satisfactory for carrying the special loading from the very heavy dumpers.

The total cost of the bridge was £65,000. The piling took four months. The pile-caps and the difficult foundations at the western and eastern ends took a further five months. The superstructure, including the western hollow-box abutment, the piers, deck construction, parapets, and the asphalt carriageway, took a further seven months.

The consulting engineers were Messrs. Oscar Faber & Partners. The main contractors were Messrs. McLaughlin & Harvey Ltd. The piles were constructed by The Cementation Co., Ltd.

Reinforced Concrete Association.

THE subject of the meeting of the Reinforced Concrete Association to be held at the Institution of Structural Engineers in London on December 6 is "Watertight Concrete Basements", by L. R. Creasy, B.Sc., M.I.C.E., M.I.Struct.E. The following is a synopsis.

The principles of successful design and construction of basements are reviewed and advice on sealing defects is given. A concrete structure can be made watertight and damp-proof without the assistance of a membrane. Generally the head of water is difficult to predict. The permissible stresses may be increased if the full head to ground level is allowed for in the design.

Aqueduct, Alloz.

Tempul Aqueduct.

Market Hall, Algeciras.

Professor Eduardo Torroja and some of his Works.

(*See facing page.*)

The Works of the late Professor Eduardo Torroja.

Dr. P. W. Abeles writes:

Professor Eduardo Torroja, whose death occurred in June last, at the age of 61, was one of the few engineers who combined a profound knowledge of the theory and behaviour of structures with architectural imagination. He was one of the first to introduce practical tests of three-dimensional models of structures. He has left, in addition to many structural creations of his design, a valuable book entitled "Philosophy of Structures" in which are described in general terms the basic purposes of structures apart from the technical and computative aspects. He can claim to have introduced prestressed concrete into his works as early as 1925 and he was one of the pioneers of concrete shell construction; his hyperbolic-paraboloidal roofs built in 1935 are famous for their daring appearance due to the extensive cantilevers. The two relative organisations of which Prof. Torroja was president, that is the Fédération Internationale de la Précontrainte and the Association of Shell Structures, have lost their leader, a loss which was particularly untimely in view of the work in hand in connection with the symposia on model testing and simplified methods of designing shells which were held in Delft and Brussels respectively in August and September of this year. Prof. Torroja was Director of the Technical Institute of Construction and Cement at Madrid, which Institute was founded by him in 1934 and in which he introduced and developed model testing to such an extent that his laboratory has become world famous in this respect.

The few examples of Prof. Torroja's work described in the following are sufficient to show his creative activity and are taken from his autobiography, "The Structures of Eduardo Torroja". Some of his early works, which are now mainly of historical interest, have been illustrated in past numbers of this journal, and show how soon he became a master in his line. Illustrations of some of these works are given on page 384.

In his first prestressed concrete work, Prof. Torroja used high-strength steel ropes (strand) of 2¼ in. diameter. Since this was over thirty-five years ago, this may be considered to be the very first concrete structure in which much of the initial prestress must have remained.

Fig. 1 shows his pioneer prestressed concrete structure, the Tempul Aqueduct, the initial design of which consisted of five equal spans of 66 ft. each, with two river piers supported on piles. Since the authorities did not approve this design, Prof. Torroja changed it by omitting the river piers and including two tie-members each of which is 135 ft. long and extends over a support and is anchored close to an outer pier. The strands, the ends of which were anchored in the concrete, were stressed by jacking them up 10 in. above the supports, thereby lifting the overhanging spans slightly, and at the same time lifting the concrete structure off the centering. Another early example of Prof. Torroja's creative constructions is a prestressed concrete aqueduct built in 1939 at Alloz (see page 384 and this journal for October 1948), a structure which is noteworthy not only because of its U-shape but also because of details of the system of forcing apart end-anchored steel members; the system was re-created in Germany under the name "Spreizen" some ten years later. In addition to the strands extending the entire length of the structure, there were pairs of short tensioned strands over the supports and, in addition to the longitudinal prestress,

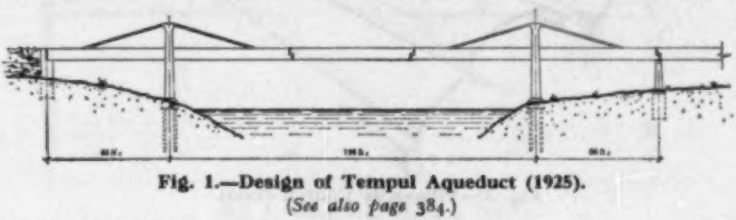


Fig. 1.—Design of Tempul Aqueduct (1925).
(*See also page* 384.)

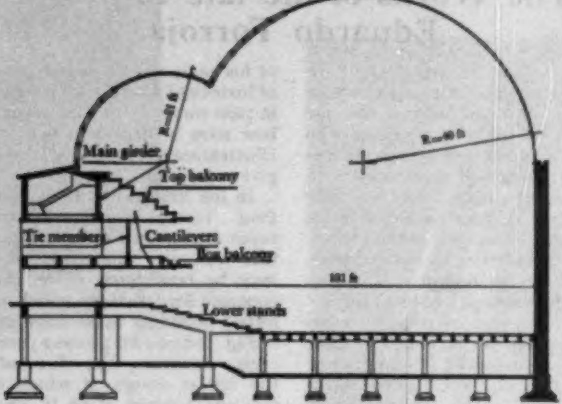


Fig. 2.—Fronton Recoletos (1935).

transverse prestress was also applied by means of turnbuckles. It has been stated that there are no signs of cracking in this aqueduct which is 1340 ft. long.

Cylindrical barrel shells had been introduced in Germany about 1924 by the Zeiss-Dywidag system, but Prof. Torroja was probably the first to develop this idea by the combination of two cylindrical shells with skylights in the roof, as at the Fronton Recoletos in 1935 (*Fig.* 2); this structure is illustrated in this journal for January 1956. For the skylights a triangulated reinforced concrete structure was provided in which the glass panels were placed. In *Fig.* 2 it is seen that there is a main girder, which is 11 ft. 6 in. deep and 72 ft. long, provided to support transverse beams from which the lower balcony is suspended. The roof, which was hit several times during the Spanish civil war, exhibited great resistance, in spite of great deformations, but it eventually collapsed due to buckling caused by progressive creep since it was impossible to shore up the structure in time and to strengthen it. Another construction of the same year is the roof of the Villaverde church (*Fig.* 3), where a central semi-elliptical shell is connected to two quarter ellipses at each side. The dome of the market at Algeciras (see page 384 and this journal for January 1956) has a diameter of 156 ft. and the shell is only 3½ in. thick. In this structure, Prof. Torroja introduced intentionally, as early as 1933, a stressed ring-beam.

The hyperbolic-paraboloidal cantilevered

Fig. 3.—Villaverde Church (1935).

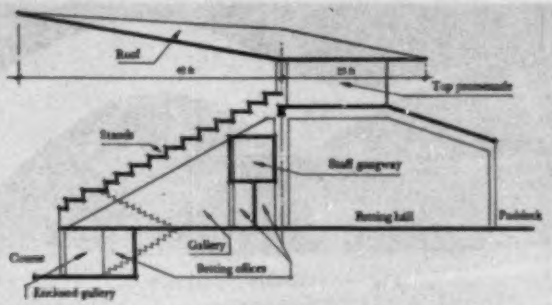


Fig. 4.—A Preliminary Scheme for the Grandstand, Madrid (1935).

roof (*Figs.* 4 and 7) of the grandstand, Madrid, built in 1935, is one of the first, if not the first, of this type of currently popular form of construction.

In ordinary reinforced concrete, the Esla bridge of 690 ft. span, built in 1939, was the largest concrete arch at the time and is still the largest railway arch bridge; the crown is 330 ft. above the ground. The arrangement of the centering, which later formed part of the reinforcement of the arch, and jacking apart at the crown are two among a number of novel features.

A further example of how he solved problems with some elegance is seen in *Fig.* 5, which shows the operating-theatre of the University City Hospital, Madrid, which was built in 1934. The roof covers a circular area of 70 ft. diameter without intermediate columns and a circular skylight of 32 ft. diameter is provided at the centre. In this design, the obvious solution was discarded and a sound and better alternative, in which ring forces are developed to counteract the cantilever action, was adopted.

Fig. 6 shows an example of Prof. Torroja's recent work, a shell roof for the Tachira Club in Caracas, Venezuela, built in 1957. This double-curved shell has a thickness of 4 in. and is supported on one side by a continuous foundation on roller bearings and by three separate supports. The arches cutting the shell are made rigid by thin ties which allow an open view. Glass panelled façades are provided to act as boundary stiffeners. Rods are provided to resist the horizontal thrust at floor level and longitudinal prestressing above the roller bearings was applied to prevent cracking. By means of the ties all horizontal forces are counterbalanced and vertical forces only are transmitted to the ground.

Since the combination of wisdom and

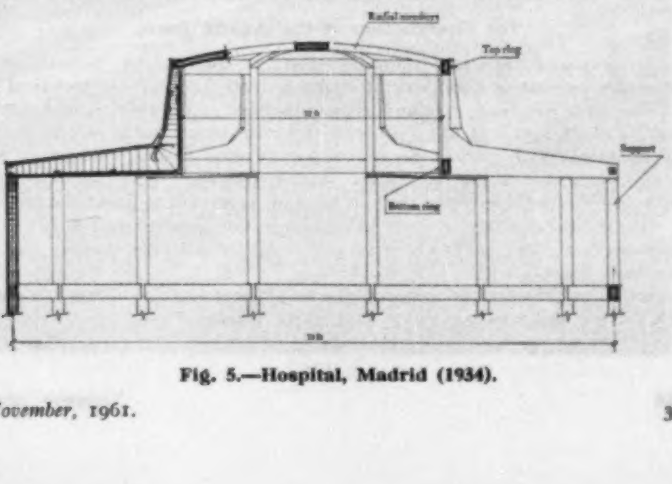


Fig. 5.—Hospital, Madrid (1934).

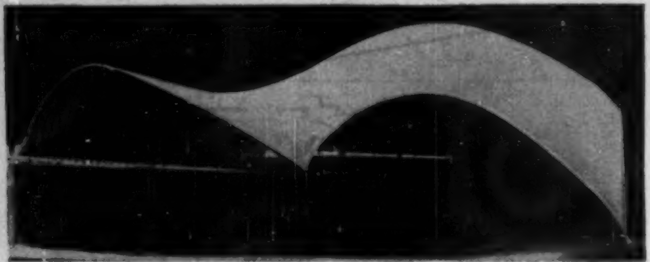


Fig. 6.—Tachira Club (1957).

Fig. 7.—Grandstand, Madrid (1935).

modesty had made him a most likable personality, Professor Torroja will be greatly missed not only by his friends but by all those who are interested in the development of new ideas and the co-ordination of theory and practice. The writer of these notes had the good fortune to be well acquainted with Prof. Torroja.

[NOTE.—*Figs.* 2 to 6 are reproduced from "The Structures of Eduardo Torroja" by permission of F. W. Dodge Corporation.]

FIFTY YEARS AGO.

From "CONCRETE AND CONSTRUCTIONAL ENGINEERING", November, 1911.

The Destruction of the Austin Dam.

"THE destruction of the Austin Dam on September 30th last has been the subject of a considerable amount of erroneous information in the popular and technical press at home. The dam has been in many cases described as of reinforced concrete. It was nothing of the sort. It was a concrete structure which had at certain points been slightly reinforced with metal. Secondly, certain other papers commented upon the accident as being due to the fact that it was of concrete. This, again, is erroneous, for the failure was not due to the body of the dam at all, but to unsatisfactory foundations. Besides, the dam had already failed to a certain extent long before the disaster of September 30th, and, owing to delay for some inexplicable reasons, the damage had not been made good.

American technical papers reiterate the fact that it was due to faulty foundations, the dam having slid forward on its base until it toppled over."

The New Grandstand at Ascot.

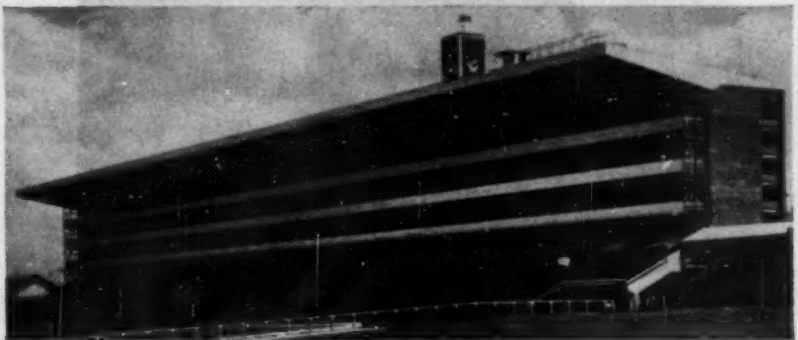


Fig. 1.

Concrete has been used n many ways and for varying purposes in the new Queen Elizabeth II stand (*Fig.* 1) at Ascot racecourse which was opened in June last. The constructional period was only ten months and the tight programme required that as many as possible of the structural components should be prepared before work commenced on the site. Consequently the floors, terraces, stairs to the boxes, balcony balustrades and some flights of main stairs are of precast concrete. The main structure and roof are of structural steelwork, the main frame being encased in concrete for fire protection and in order to reduce maintenance. The grandstand comprises two main parts. The principal section, which faces the racecourse, is 560 ft. long and 80 ft. wide, and has an average height of 74 ft. Centrally at the rear of this section there is a circulation area one third as long. At the front of the main section, the terrace (*Figs.* 2 and 3), which accommodates 8000 persons standing, rises in steps at a slope of 1 in 2.

Fig. 2.

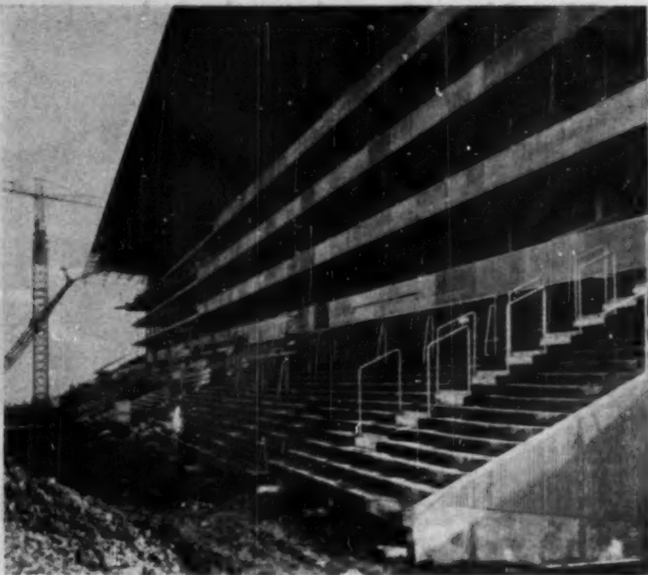


Fig. 3.

Precast Construction.

The steps of the terrace are constructed of hollow precast reinforced concrete units (*Fig.* 4) and, because of the pedestrian traffic to which they are subjected, the slab over the cavity is 1½ in. thick. For rigidity, a span of 10 ft. has been adopted for this section, which is half the basic span elsewhere throughout the structure. The steps overlap one another, with the overlap bedded and pointed in mortar, and are coated with asphalt. Behind the terraces is an area occupied by stall seats supported on a continuation of the raking beams of the terrace. The seats are bolted to ell-shaped precast reinforced concrete units each forming the going and the riser of a step (*Fig.* 5). These units are of concrete having a strength of 4000 lb. per square inch at twenty-eight days. The balustrade in front of the stalls is of concrete precast in panels and ribbed vertically on the outer face (*Fig.* 6), which pattern is repeated for most of the vertical panels throughout the grandstand.

The main structure behind the terraces comprises seven main floors which are of suspended construction and include ground and mezzanine floors, and five private dining-room floors, which serve the boxes on three balconies. The middle dining-room floor is at the same level as the adjacent balcony, but access from the remaining upper and lower dining-room floors to the boxes is by means of

Fig. 4.

Fig. 5.—Precast Seating.

short flights of steps. The main floor slabs spanning between the steel beams are prestressed hollow precast units (*Fig.* 4), 3 ft. 2 in. wide and 7½ in. deep; they contain cables of pretensioned 0·2-in. crimped wires and span 20 ft. The slabs, in each of which are three cavities, were made by the long-line method. Since the soffits were painted after erection, the units were cast in special steel moulds with bold arrises. The crushing strength of cubes of the concrete for these slabs was 6000 lb. per square inch at twenty-eight days, the strength at transfer being 4000 lb. per square inch. The imposed design load is 100 lb. per square foot throughout the floors except in the private dining rooms and boxes where it is 80 lb. per square foot. Load tests and handling tests were carried out on prototypes of the units. To reduce the storey heights, the floor beams are laid with their tops 1 in. above the top of the supporting steel beams and are seated on rebates in the concrete encasing the beams. The width of the seating of the slabs is 2½ in. since it was necessary to insert the slabs under the top flanges of the beams and, because of this, to allow for a tolerance in positioning the slabs. The total area of precast flooring and terraces is 17,000 sq. yd. which requires 4600 units. The units were cast to very close dimensional tolerances.

The balcony balustrade (*Fig.* 6) extends the full length of the grandstand and comprises 168 precast ribbed panels each 10 ft. long. The panels were lifted into place by being slung from gear engaging with bolts in the top edge (*Fig.* 7); the bolts were afterwards used to attach a hardwood cap-rail. The panels were temporarily maintained in a vertical position by two tapped studs welded to the top flange of the steel channel which supported them. The permanent fixing of the panels to resist bending was effected by reinforcing bars protruding horizontally from the panels and embedded in the 2-in. structural topping on the first two precast slabs of the balcony floor; these two slabs differed from the ordinary slabs for the balcony, in so far that they are

Fig. 6.—Precast Balustrade.

Fig. 7.—Lower Box-balcony.

prestressed hollow beams 1 ft. 9 in. wide with a key formed between them to provide a mechanical bond with the topping. The rear balconies giving access to the dining-room floors have balustrade panels similar to those of the box balconies, but greater in height, and are fixed in a similar manner.

Included in the precast construction are the five different types of units forming parts of the access from the dining rooms to the boxes at the split-levels (*Fig.* 8). In each 20-ft. bay these units comprise four stairs (two up and two down), solid panels alongside the downward flights, and panels filling the triangular space between the flights; the latter panels are in two pieces for ease of handling. The concrete of the stairs contains granite aggregate and has a finish of carborundum trowelled into the treads.

It was necessary to stockpile all precast units before work commenced on the site in order to avoid delays.

Encased Steelwork.

The concrete encasing the steelwork provides a minimum cover of 2 in. over the steel. The casing of the stanchions (*Fig.* 9) is generally reinforced with high-tensile welded fabric weighing 1 lb. per square yard, but is increased to comply with the requirements of B.S. No. 449 where the concrete casing is taken into account in assisting the load-bearing capacity of the encased stanchions, as is necessary in a few cases where it is desirable to have the stanchions all of the same section. The concrete for the casings is a nominal 1 : 2 : 4 mixture with a ¾-in. aggregate, and has a strength of 3000 lb. per square inch at twenty-eight days; the proportions were selected after tests of workability and strength. The concrete encasing the steel beams is reinforced and secured by fabric similar to that used for the stanchions. Where the casing envelops only the lower part of a beam, ½-in. diameter mild steel bars pass through holes drilled in the web of the beam to secure the reinforcement and the casing.

Cast-insitu Construction.

In addition to the large extent of precast work there are some parts, mainly stairs and landings, which are of ordinary cast-insitu construction generally of 1 : 1½ : 3 reinforced concrete.

One of the two pairs of escalators is external and is contained in a reinforced

Fig. 8.—Stairways of Precast Units.

Fig. 9.—Encased Steelwork supporting Prestressed Concrete Floors.

concrete trough (*Fig.* 11) which also acts as the beam to support the stair and spans between the levels of the ground and mezzanine floors. This construction also carries parallel stairs which cantilever from it on either side. The concrete for this work is 1 : 1½ : 3 with ¾-in. aggregate.

Foundations.

The foundations are in the sand and sandy gravel of the Bagshot series and are at a relatively shallow depth. The net bearing pressure is 1¾ tons per square foot. The main foundations, where separate from those of the walls, are in the form of independent pads and continuous strips. The independent pads are of ordinary reinforced concrete design and comprise 1 : 2 : 4 concrete with 1½-in. aggregate and are laid on a 1 : 12 blinding concrete. The largest pads are 13 ft. 6 in. square. The continuous strip foundations are 8 ft. wide by 6 ft. deep and are of 1 : 10 concrete with 1½-in. aggregate. Construction joints were formed at the third-points between the stanchions to suit the amount of concrete placed in one operation. High-tensile fabric weighing 10·76 lb. per square yard is provided in the bottom. At the position of each stanchion, a pocket was formed to receive a pad of 1 : 2 : 4 concrete which acts as a seating for the steelwork and contains the holding-down bolts; to distribute the loading from these pads, 1-in. diameter mild steel bars are provided in the concrete strip footing below the pockets.

Other foundations in the circulation area are combined one with another and with the lift pits and the retaining walls

Fig. 10.—Construction of Basement Floors.

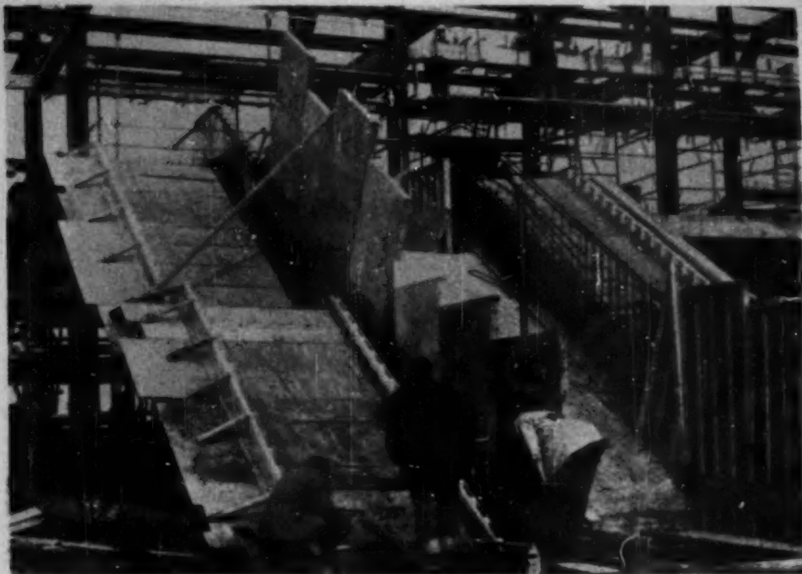


Fig. 11.—Cast-insitu Construction of Escalator Supports and Stairs.

in this area, where a basement is provided. The stanchions form piers for the walls which span between them. Because of the low water-table, no tanking was applied to the basement walls which are reinforced with high-tensile fabric and constructed in 1 : 1½ : 3 concrete. P.V.C. water-bars are provided in all construction joints.

Work in progress on the construction of the basement slabs, which are laid on sheets of polythene on the ground, is shown in *Fig.* 10.

The main contractors were Messrs. George Wimpey & Co., Ltd., who were also responsible for the structural design. Sub-contractors for the precast concrete work were Concrete (Southern) Ltd., for the units for the floors and terraces, and Messrs. Cawood Wharton & Co., Ltd., and Messrs. John Ellis & Sons, Ltd., for stalls, stairs and panels. The two tower-cranes were Pingon cranes.

The information in the foregoing is contributed by Mr. D. Dennington, B.Sc., A.M.I.C.E.

Bulletins Received.

"**Country Roads Board.—Forty-Seventh Annual Report.**" (Melbourne, Australia: 1961. No price stated.)

"**Co-operation Between Main Contractors and Sub-Contractors in the Building Industry.**" (London: National Federation of Building Trades Employers. 1961. Price 2*s.* 6*d.*)

"**The Professional Engineer—His Employment and Development.**" (London: The Engineers' Guild Ltd. 1961. Price 7*s.* 6*d.*)

The following documents are published by H.M.S.O. (London, 1961) for the Ministry of Labour.

"**Factories Acts, 1937-1959 Report—Lifting and Construction Regulations.**" (Price 3*s.*)

"**Factories.—The Construction (General Provisions) Regulations No. 1580, 1961.**" (Price 9*d.*)

"**Factories.—The Construction (Lifting Operations) Regulations No. 1581. 1961.**" (Price 1*s.*)

Permissible Stresses in Road Bridges.

THE Ministry of Transport have announced that the permissible working stresses in concrete and reinforcing bars in road bridges and structures have been revised by the Bridges Committee of the Road Research Board and are accepted by the Ministry as being applicable to designs which require the Ministry's approval. The revised stresses, which are to be used in conjunction with the loads described in Memorandum No. 771,* are given in a new Memorandum No. 785, "Permissible Working Stresses in Concrete and Reinforcing Bars for Highway Bridges and Structures". The recommendations are in a form suitable for a code of practice, and their adoption necessitates replacement of certain clauses in the Ministry's Standard Specification. The sections of Memorandum No. 577, "Bridge Design and Construction", relating to concrete are now superseded.

Permissible Stresses in Reinforcement.

LOADING TYPE HA. (B.S. No. 153, Part 3, Section A—1954).—In mild steel bars complying with B.S. No. 785, the stresses in tension and compression should not exceed 18,000 lb. per square inch for bars of all sizes. In cold-twisted steel bars complying with B.S. No. 1144 and for deformed bars of high-tensile steel with a guaranteed yield-stress, the stress in tension should not exceed half the guaranteed yield-stress or 30,000 lb. per square inch, whichever is less. The stress in compression should not exceed half the guaranteed yield-stress nor be more than 23,000 lb. per square inch.

CRACK CONTROL.—When the reinforcement is of steel other than mild steel and when the stress exceeds 18,000 lb. per square inch in tension, the arrangement of bars determined from the stress calculations should be checked for crack control by applying the formula

$$T = \frac{n l_a E_s \sqrt{D}}{Z f_s},$$

where

n = number of bars in tension in the section; where bars in slabs are spaced uniformly, n is equal to 12 in. divided by the distance (inch) between centres of adjacent bars,

l_a = lever arm (inch) measured to the centroid of the reinforcement,

E_s = modulus of elasticity of the steel (lb. per square inch),

Z = section modulus (in.^{3}) of the whole section with regard to the edge which is in tension, as if it were plain concrete; for slabs, a width of 12 in. is taken,

f_s = stress (lb. per square inch) in reinforcement at cracked section, at the centroid of the reinforcement,

D = diameter of the reinforcing bar (inch).

Minimum acceptable values of T (in inch units) are as follows.

	Deformed Bars	Cold-twisted Bars
Dead load . .	105	140
Dead load + live load . . .	70	90

If bars in beams are of different sizes, the calculations should be carried out for each group of bars of the same size and the values of T added together.

LOADING TYPE HB. (B.S. No. 153, Part 3, Section A—1954).—The maximum stresses permitted for loading type HA may be increased by 25 per cent., but should not exceed 30,000 lb. per square inch in tension or 23,000 lb. per square inch in compression.

Permissible Stresses in Concrete.

The compressive, shearing and bond stresses in reinforced concrete should not exceed those given in *Table* I. The permissible bond stresses may be increased by 25 per cent. when cold-twisted bars and deformed bars are used. When the proportions of total aggregate to cement are between those given in *Table* I, the permissible concrete stresses should be based on the ratio of the sum of the volumes of the fine and coarse aggregate, measured separately, to the quantity of cement and should be obtained by proportion from the two nearest nominal proportions.

* Memorandum No. 771, "Standard Highway Loads".

TABLE I

Nominal proportions			Aggregate per 112 lb. of cement (cu. ft.)		Cube strength within 28 days after mixing (lb. per sq. in.)		Cube strength within 7 days after mixing (lb. per sq. in.)		Permissible concrete stresses (lb. per sq. in.)				
									Compression		Shearing	Bond	
Cement (lb.)	Fine Aggregate (cu. ft.)	Coarse Aggregate (cu. ft.)	Fine	Coarse	Preliminary test	Works test	Preliminary test	Works test	Direct	Due to bending		Average	Local
150	2	4	1½	3	5600	4200	3730	2800	1060	1400	125	145	210
120	2	4	1⅞	3¾	5000	3750	3350	2500	950	1250	115	135	200
90	2	4	2½	5	4000	3000	2700	2000	760	1000	100	120	180

When considering the effect of loading type HB, it is permissible to allow an overstress of 25 per cent.

Consideration will be given to the use of higher working stresses in special cases provided there are adequate safeguards to ensure sufficiently close control of the quality and workmanship of the concrete.

References to B.S. No. 153, Part 3, Section A (1954), should be read in conjunction with Ministry of Transport Memorandum No. 771.

International Association for Bridge and Structural Engineering.

PAPERS of interest to reinforced concrete engineers published in the twentieth volume (1960) of the "Publications" of the International Association for Bridge and Structural Engineering, include the following.

"A General Analysis of Elasto-Plastic Three-Dimensional Frames." By A. L. L. Baker (Great Britain; in English).

"Theory of Shells of any Shape." By E. Bolcskei (Hungary; in German).

"Calculation of Groups of Piles with a Non-Linear Relationship Between Force and Penetration." By E. Gruber (Germany; in German).

"Curved Edge Disturbances in Circular Cylindrical Shells." By I. Holand (Norway; in English).

"Spherical Domes Under Unsymmetrical Loading. By T. van Langendonck (Brazil; in English).

"The Vibrations of Massive Foundations on Soil." By M. Novak (Czechoslovakia; in English).

"Membrane Stresses in Hyperbolic Paraboloid Shells Circular in Plan." By E. P. Popov and S. J. Medwadowski (U.S.A.; in English).

"Elasto-Plastic Analysis of an Interconnected Beam System." By D. V. Reddy and A. W. Hendry (Great Britain; in English).

"Abnormal Loading on Two Types of Short Span Bridge." By R. E. Rowe (Great Britain; in English).

"Strength Calculation of Reinforced Concrete Structures." By Y. Saillard (France; in French). Preparation of standards; Report on the work of the 5th Meeting of European Committee on Concrete.

"On the Relaxation of Steel Wires." By F. Speck (Switzerland; in German).

Copies of "Publications of the International Association for Bridge and Structural Engineering," Volume 20, 1960 (price 45 Swiss francs), are obtainable from Verlag Leemann, Zürich. "Publications" contains eighteen papers of which twelve are in English, two in French and four in German. Each paper is followed by a summary in each of these languages.

The Design of Doubly-reinforced Flanged Beams by the Load-factor Method.

By YÜKSEL SEZGINER, Dipl. Ing. (Turkey).

THE nomogram on pages 398 and 399 applies to the design of doubly-reinforced beams of tee, ell or I section by the load-factor method, and is based on the following equations.

$$\frac{M_r}{p_{cb}bd_1^2} = \frac{b_r}{4b} + \frac{1}{3}\left(1 - \frac{b_r}{b}\right)\left[2\frac{d_s}{d_1} - \left(\frac{d_s}{d_1}\right)^2\right] + R_{sc}r_{sc}\left(1 - \frac{d_2}{d_1}\right) \quad . \quad (1)$$

$$r_{st}R_{st} = \frac{2}{3}\left[\frac{d_s}{d_1} + \frac{b_r}{b}\left(\frac{1}{2} - \frac{d_s}{d_1}\right)\right] + r_{sc}R_{sc} \quad . \quad . \quad . \quad . \quad (2)$$

in which $r_{sc} = \frac{A_{sc}}{bd_1}$, $R_{sc} = \frac{p_{sc}}{p_{cb}}$, $r_{st} = \frac{A_{st}}{bd_1}$, and $R_{st} = \frac{p_{st}}{p_{cb}}$. The use of the nomogram is explained by the examples which follow.

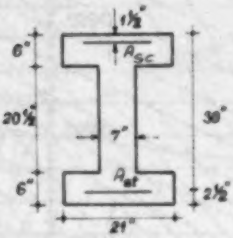


Fig. 1.

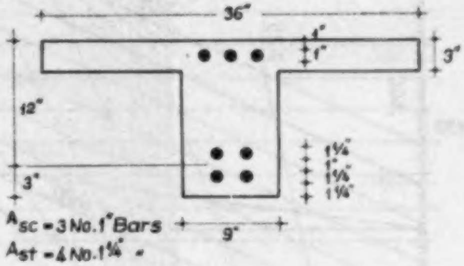


Fig. 2.

EXAMPLE I.—Determine the amount of reinforcement required in the I-beam in *Fig.* 1 if it is subjected to a bending moment of 4,725,000 in.-lb. The permissible stresses are $p_{cb} = 1000$ lb., $p_{st} = 20{,}000$ lb., and $p_{sc} = 18{,}000$ lb. per square inch.

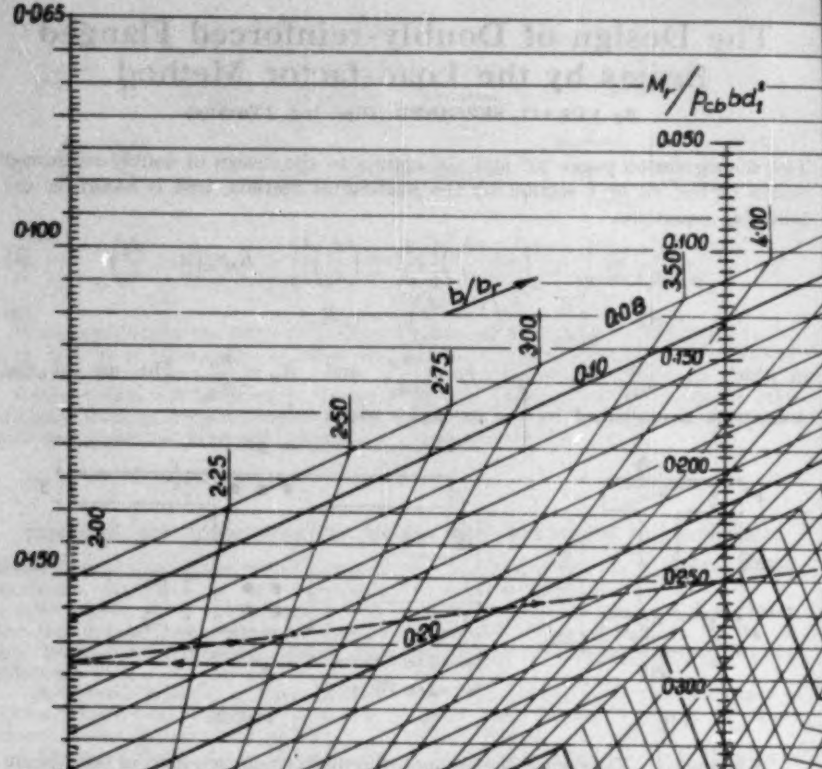
First step.—Calculate the ratios for use with the nomogram, namely,

$$\frac{b}{b_r} = \frac{21}{7} = 3; \ \frac{d_s}{d_1} = \frac{6}{30} = 0{\cdot}20; \ \frac{d_2}{d_1} = \frac{1{\cdot}5}{30} = 0{\cdot}05; \ R_{sc} = 18;$$

$$\frac{M_r}{p_{cb}bd_1^2} = \frac{4{,}725{,}000}{1000 \times 21 \times 900} = 0{\cdot}250; \text{ and } R_{st} = 20.$$

Second step.—Intersect the vertical line for $\frac{b}{b_r} = 2{\cdot}0$ at 0·163 by a horizontal line drawn from the intersection of the lines for $\frac{b}{b_r} = 3$ and $\frac{d_s}{d_1} = 0{\cdot}20$.

Connect the point for 0·163 with $\frac{M_r}{p_{cb}bd_1^2} = 0{\cdot}250$ and produce the line until it cuts the vertical line for $\frac{d_2}{d_1} = 0$ at 0·087. A horizontal line drawn from this point to intersect the line for $\frac{d_2}{d_1} = 0{\cdot}05$ at $R_{sc}r_{sc} = 0{\cdot}092$. Produce it on to the



$M_r / p_{cb} b d_1^2$
b/b_r
d_s/d_1
d_2/d_1
d_2/d_1
0·065
0·100
0·150
0·200
0·250
0·050
0·100
0·150
0·200
0·250
0·300
0·350
0·400
0·450
0·500
0·550
2·00
2·25
2·50
2·75
3·00
3·50
4·00
0·08
0·10
0·20
0·30
0·40
0·50
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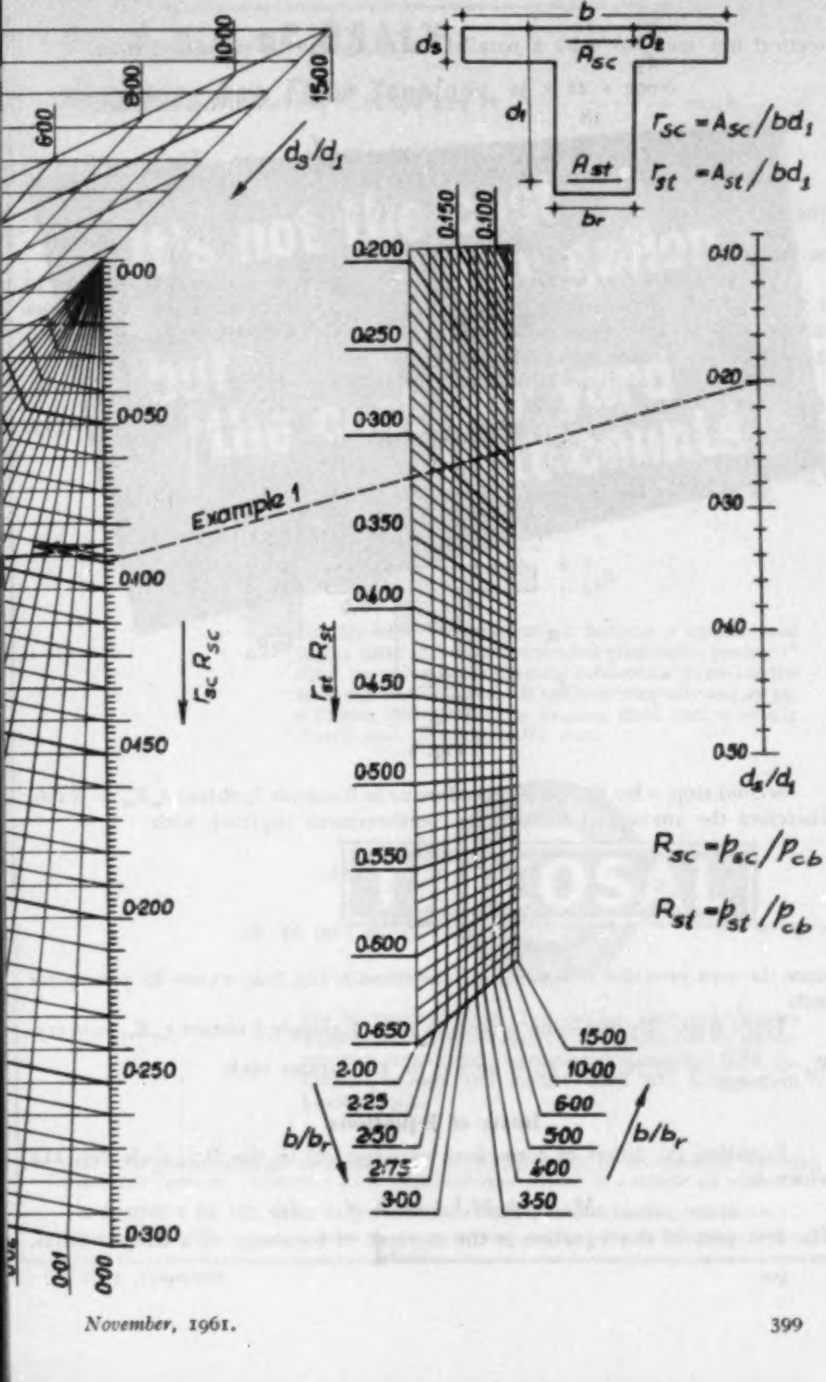
0·35
0·30
0·25
0·20
0·15
0·10
0·09
0·08
0·06
0·05


b
d_s
d_2
A_{sc}
d_1
A_{st}
b_r
$r_{sc} = A_{sc}/bd_1$
$r_{st} = A_{st}/bd_1$
10·00
8·00
6·00
15·00
d_s/d_1
0·00
0·050
0·100
0·150
0·200
0·250
0·300
Example 1
r_{sc} R_{sc}
0·01
0·02
0·150
0·100
0·200
0·250
0·300
0·350
0·400
0·450
0·500
0·550
0·600
0·650
r_{st} R_{st}
2·00
2·25
2·50
2·75
3·00
3·50
4·00
5·00
6·00
10·00
15·00
b/b_r
b/b_r
0·10
0·20
0·30
0·40
0·50
d_s/d_1
$R_{sc} = p_{sc}/p_{cb}$
$R_{st} = p_{st}/p_{cb}$

vertical line for $\frac{d_s}{d_1} = 0$ by a parallel line. A_{sc} can be calculated from

$$A_{sc} = \frac{0{\cdot}092 \times 21 \times 30}{18} = 3{\cdot}22 \text{ sq. in.; provide five 1-in. bars.}$$

Third step.—Connect 0·092 with the point for $\frac{d_s}{d_1} = 0{\cdot}20$. The line intersects the vertical for $\frac{b}{b_r} = 3$ at $r_{st}R_{st} = 0{\cdot}292$. Then $A_{st} = \frac{0{\cdot}292 \times 21 \times 30}{20} = 9{\cdot}20$ sq. in.; provide six $1\frac{1}{8}$-in. bars.

EXAMPLE II.—The tee-beam in *Fig.* 2 is subjected to a bending moment of 1,375,000 in.-lb. Assuming $p_{cb} = 1000$ lb. per square inch, check that the stresses in the tension and compression reinforcement do not exceed 30,000 lb. and 23,000 lb. per square inch respectively.

First step.—Calculate the necessary ratios for use with the nomogram, namely, $\frac{b}{b_r} = 4$, $\frac{d_s}{d_1} = \frac{3}{12} = 0{\cdot}25$; $\frac{M_r}{p_{cb}bd_1^2} = \frac{1{,}375{,}000}{1000 \times 36 \times 144} = 0{\cdot}265$; $\frac{d_2}{d_1} = \frac{1{\cdot}5}{12} = 0{\cdot}125$; $r_{st} = \frac{4{\cdot}91}{36 \times 12} = \frac{1{\cdot}137}{100}$; and $r_{sc} = \frac{2{\cdot}36}{36 \times 12} = \frac{0{\cdot}546}{100}$.

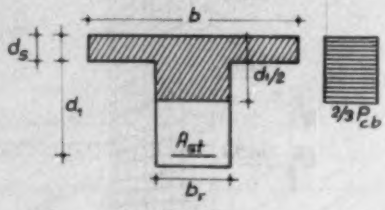


Fig. 3.

Second step.—By the same procedure as in Example I, obtain $r_{sc}R_{sc} = 0{\cdot}106$. Therefore the amount of compression reinforcement required, with

$$R_{sc} = \frac{23{,}000}{1000} = 23,$$

is given by
$$A_{sc} = \frac{0{\cdot}106 \times 36 \times 12}{23} = 1{\cdot}99 \text{ sq. in.;}$$

since the area provided is 2·36 sq. in., the stress is less than 23,000 lb. per square inch.

Third step.—By the same procedure as in Example I obtain $r_{st}R_{st} = 0{\cdot}315$, $R_{st} = \frac{31{\cdot}5}{1{\cdot}137} = 27{\cdot}70$, and $p_{st} = 27{,}700$ lb. per square inch.

Basis of Equations.

Equation (1) differs in form from equation (6) in the B.S. Code No. 114 which is

$$M_r = \gamma p_{cb}bd_1^2 + A_{sc}p_{sc}(d_1 - d_2).$$

The first part of this equation is the moment of resistance of a flanged beam,

with tension reinforcement only, based on the compressive resistance of the concrete. The force in the reinforcement in this case has to be balanced by the resistance of the concrete (*Fig.* 3). If the cross-sectional area of this reinforcement is A_{st_1} the following formula can be derived.

$$A_{st_1}p_{st} = \tfrac{2}{3}p_{cb}\left[(b-b_r)d_s + b_r\frac{d_1}{2}\right]$$

from which
$$\frac{A_{st_1}p_{st}}{p_{cb}bd_1} = r_{st_1}R_{st} = \frac{2}{3}\left[\frac{d_s}{d_1} + \frac{b_r}{b}\left(\frac{1}{2} - \frac{d_s}{d_1}\right)\right].$$

In the case of a beam with compression reinforcement, the total amount of tension reinforcement is $A_{st} = A_{st_1} + A_{sc}\dfrac{p_{sc}}{p_{st}}$. Also $r_{st}R_{st} = r_{st_1}R_{st} + r_{sc}R_{sc}$. From the preceding equation, $r_{st}R_{st} = \dfrac{2}{3}\left[\dfrac{d_s}{d_1} + \dfrac{b_r}{b}\left(\dfrac{1}{2} - \dfrac{d_s}{d_1}\right)\right] + r_{sc}R_{sc}$, which is equation (2).

"Effects of Differential Temperature on Tall Slender Columns."

Mr. A. J. Ashdown writes as follows.

Since I contributed an article on thermal stresses in this journal for August 1933, the article by Mr. D. A. Stephenson in the number for May 1961 is of particular interest to me. The suggestion that the temperature through the concrete varies exponentially is new to me and it would be interesting to know how the factor $a = \sqrt{\tfrac{1}{2}\omega CR}$ is derived; the effect of dispersion of heat through the ends is not included. There seems to be a great difference between a linear variation and an exponential variation of thermal strain, especially so for thick concrete members. Experimental and analytical evidence for assuming an exponential variation would be of great interest, as recent experiments carried out at the Imperial College, London, indicate a perfectly linear variation of temperature through concrete 13 in. thick, with the sides of the specimen insulated against heat loss. If the assumption of an exponential variation is valid, the assumption that the moments due to heat-strain and linear strain about the cool face are equal is convenient but is difficult to justify.

If plane sections remain plane after the imposition of an exponential distribution of heat-strain, internal stresses will be set up. In order to determine these stresses, two conditions must be satisfied, namely, that there is no difference of strain between the exponential heat-strain and a linear strain, and that the moments caused by these strains and acting about the centroid of the section should balance. The second condition will cause a strain αT_n to act at the cool face. The heat-strain at any point at a distance x from the hot face is $\alpha T_0 e^{-ax}$. The equivalent linear strain is

$$\alpha T_n + \alpha(T_1 - T_n)\left(1 - \frac{x}{d}\right).$$

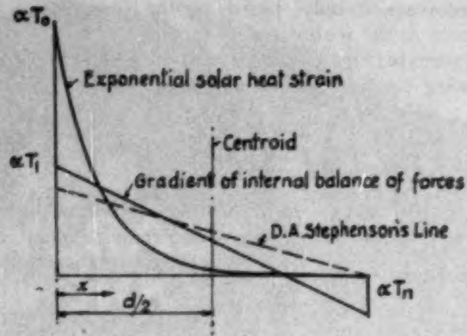
The difference of strain is

$$\alpha T_0 e^{-ax} - \alpha T_n - \alpha(T_1 - T_n)\left(1 - \frac{x}{d}\right) \quad . \quad . \quad . \quad . \quad (a)$$

Then from the first condition, the integral of expression (a) gives the total difference of strain which is

$$\alpha T_0\left(\frac{1}{a} - \frac{e^{-ad}}{a}\right) - \alpha T_n d - \alpha(T_1 - T_n)\frac{d}{2} = 0, \quad . \quad . \quad . \quad (b)$$

whence
$$T_1 - T_n = \frac{2T_0}{d}\left(\frac{1}{a} - \frac{e^{-ad}}{a}\right) - 2T_n \quad . \quad . \quad . \quad . \quad (c)$$



Also from the second condition

$$\int_0^d \alpha T_0 e^{-ax}\left(\frac{d}{2}-x\right) - \alpha T_n\left(\frac{d}{2}-x\right) - \alpha(T_1 - T_n)\left(\frac{d}{2}-\frac{3}{2}x+\frac{x^2}{d}\right)dx = 0,$$

from which $$\alpha T_0\left(\frac{e^{-ad}}{a^2}+\frac{de^{-ad}}{2a}+\frac{d}{2a}-\frac{1}{a^2}\right) - \alpha(T_1-T_n)\frac{d^2}{12} = 0.$$

Substituting the value of $T_1 - T_n$ from equation (c) and reducing,

$$\alpha T_0\left(\frac{e^{-ad}}{a^2}+\frac{2de^{-ad}}{3a}+\frac{d}{3a}-\frac{1}{a^2}\right)+\frac{\alpha d^2 T_n}{6} = 0 \quad . \quad . \quad . \quad \text{(d)}$$

In the example given in the article, this expression reduces to $\frac{T_0}{6}+\frac{6{\cdot}25T_n}{6} = 0$, hence $T_n = -0{\cdot}16T_0$, and from equation (c),

$$T_1 + 0{\cdot}16T_0 = \frac{2}{ad}T_0 + 0{\cdot}32T_0.$$

Thus $$T_1 = (0{\cdot}267 + 0{\cdot}16)T_0 = 0{\cdot}427T_0.$$

The stress at any point is

$$E\alpha T_0\left[e^{-ax} + 0{\cdot}16 - 0{\cdot}587\left(1-\frac{x}{2{\cdot}5}\right)\right] \quad . \quad . \quad . \quad \text{(e)}$$

Positive results indicate compressive stresses and the distribution is shown in the accompanying diagram.

The radius of curvature is

$$\frac{d}{\alpha(T_1 - T_n)} = \frac{2{\cdot}5}{(0{\cdot}427 + 0{\cdot}16)12 \times 5{\cdot}5 \times 10^{-6}} = 64{,}500 \text{ ft.}$$

The deflection at the column-head due to free curvature is

$$\frac{120^2}{2 \times 64{,}500} = 0{\cdot}114 \text{ ft.} = 1{\cdot}34 \text{ in.}$$

The maximum restraint stresses, adopting the value for E given in the article, are ± 160 lb. per square inch, to which must be added the stresses found from expression (e). The stress at the hot face is $(0{\cdot}573 \times 12 \times \alpha E) + 160 = 208 + 160 = 368$ lb. per square inch, and at the cool face is $(0{\cdot}160 \times 12 \times \alpha E) - 160 = 58 - 160 = -102$ lb. per square inch. Since the column is in direct compression, probably no tensile stresses occur.

There is an error in the integration of formula (1) which should be

$$T_0\alpha\left[\frac{e^{-ad}}{a^2}+\frac{d}{a}-\frac{1}{a^2}\right]$$

and should be repeated in formula (2). Although in the example the correct numerical solution is obtained, the expression should read

$$12\alpha \int_0^{2\cdot5} e^{-3x}(2\cdot5 - x)dx = 12\alpha\left[\frac{e^{-7\cdot5}}{9} + \frac{2\cdot5}{3} - \frac{1}{9}\right] = 12\alpha \times \tfrac{13}{18}.$$

The assumption of an initial value of E of $5\cdot5 \times 10^6$ seems rather high; a value of $4\cdot2 \times 10^6$ for concrete with a cube-strength of 6000 lb. per square inch is more correct. Prestressing concrete may not alter the modulus of elasticity but high prestress may lower its initial value.

Mr. D. A. Stephenson replies as follows.

The basic theory of heat flow does, of course, assume a linear variation of temperature through a conductor, since the rate of heat transmission, which must be constant along its path, is proportional to the temperature gradient. The point that Mr. Ashdown appears to have missed is that in practice all conductors have thermal capacity and, because of this, the temperature gradient does not become uniform until conditions of equilibrium have been achieved, which is after an appreciable lapse of time. If the temperature at the surface fluctuates during this period, the variations of temperature within the conductor, both with time and with distance from the surface, are more complex than in the idealised case upon which the principles of thermal conduction are based. For example, it would be possible to raise the surface temperature of a thick concrete wall to several hundred degrees C. almost instantaneously by allowing numerous blow-lamps to heat it, but it would not be expected that the temperature at the middle of the wall would rise equally quickly to one half of the surface temperature.

For the theoretical justification of the heat-flow formulæ, reference should be made to a text-book such as ' Elements of Heat Transfer ' by Jacob and Hawkins and the experimental work carried out in Kuwait by the Building Research Station of D.S.I.R.

I agree that my assumption of zero strain at the cool face is inaccurate, and thank Mr. Ashdown for drawing attention to this matter. I also agree that the second term within the square bracket in formulæ (1) and (2) should be $+\frac{d}{a}$.

The value of E used in the example is based on that used in the case of the main columns of the viaduct of the Tasman Bridge now being constructed at Hobart, Tasmania. A value of $5\cdot5 \times 10^6$ was adopted after careful collation of experimental data and relates to concrete having a specified minimum cube-strength of 6500 lb. per square inch at twenty-eight days. The figure suggested by Mr. Ashdown corresponds to an American formula $E = (1\cdot8 \times 10^6) + 390u$, which is generally recognised as giving low values.

Lectures on Building.

The following lectures have been arranged by the Ministry of Works. Admission is free. The meetings commence at 7.15 p.m. unless stated otherwise.

Prevention of Accidents in the Building Industry. By J. A. Hayward. Technical College, Bolton. November 16; Technical College, Connahs Quay. November 21, 7 p.m.; Technical College, Wrexham. November 22, 7 p.m.; and the Westminster Hotel, Rhyl. November 23.

New Techniques in Building. By D. Bishop. Highbury Technical College, Anglesea Road, Portsmouth. November 20.

Fire Resistance of Concrete Structures. By N. R. Hollington. Technical College, Southampton. November 21.

Work Study Aids Builders. By B. D. Wakeford. Technical College, Waterdale, Doncaster. November 22; and Technical College, Wakefield. November 23.

Work Study in the Building Industry. By K. C. M. Symons. Technical Institute, Bognor Regis. November 29.

Good Concrete From Local Aggregates. By R. Cameron. Y.M.C.A., Newcastle-upon-Tyne. November 30.

Box-frame Construction in Sweden.

To reduce cost and amount of labour required, a method of construction utilising a concrete box-frame is being used increasingly in Sweden. Although used mainly for residential structures it is said to be adaptable to any structure where repetition occurs. The illustration in *Fig.* 2 is of a students' hostel at Lund.

Standard shuttering of room-size for casting the floors and internal cross-walls is assembled on the ground and hoisted into position by crane. The shuttering is faced with plastic-faced plywood and is fitted with the necessary bolts, clamps and jacks; there is also a platform from which the concrete is placed to form a monolithic box-frame structure. The shuttering is easily removed for further use. Heating coils are embedded in the floor slab during construction and other services are incorporated in the floors and walls. Interior fittings are bolted to threaded sleeves embedded in the concrete. Items such as chutes, ducts and

Fig. 1.

Fig. 2.

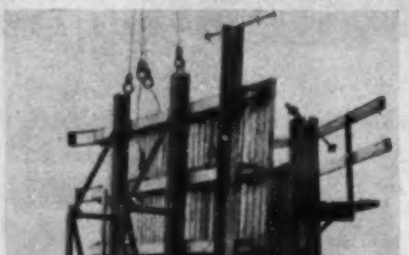


Fig. 3.

stairs are placed in position by a crane before the shuttering is re-erected for a further storey. *Fig.* 1 shows a panel of shuttering being placed in position by means of a crane. Details of the panels and the working platform attached thereto are shown in *Fig.* 3. The structures shown in *Figs.* 1 and 3 are at Edsburg.

A feature of the method of construction is that finishing work proceeds simultaneously with construction. Floor slabs are screeded, finished and polished by a machine before the next lift of the walls is cast. The plastic-faced shutters leave a surface which can be painted or wall-papered after a simple polishing operation. Exterior walls are usually of precast concrete panels 3½ in. thick, and sandwiched between this facing and the interior finish of plasterboard backed by aluminium foil is a 4-in. insulating layer of rock-wool. Precast panels of various finishes are used or alternatively brick infilling is used. An

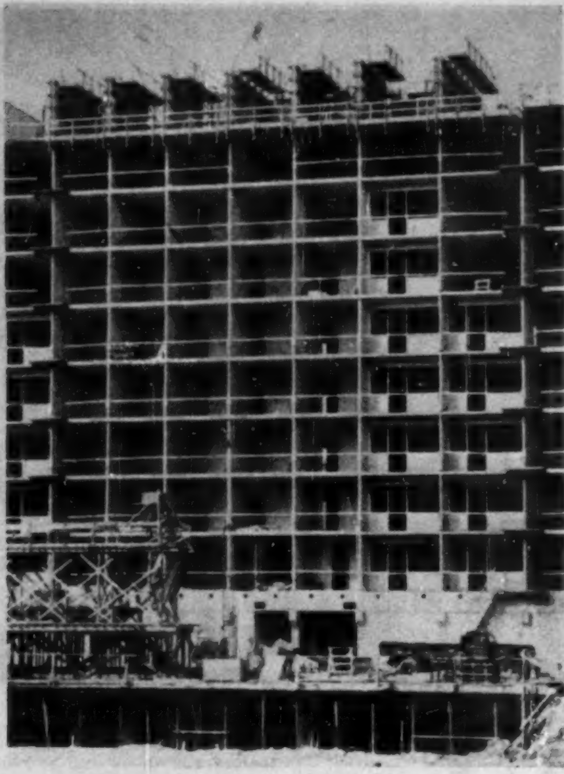


Fig. 4.

example of the latest applications is the construction of residential buildings in Malmö which include five buildings with from five to twenty-five storeys, containing shops, offices, tennis courts and a large garage in addition to upwards of seven hundred flats. The illustration in *Fig.* 4 is of one of these multiple-storey buildings in the course of construction.

AB Skånska Cementjuteriet, a firm of Swedish contractors, developed the method, which is known as the "Allbetong" method, in the early 1950's and claim that by its use the number of man-hours required for a given volume of building has been reduced by about half compared with conventional constructional methods.

Ready Reckoners.

REVISED British Standard B.S. 1151 (Part 2—1961), "Guaranteed Minimum Reckoners for the Building and Civil Engineering Contracting Industries", was published recently. The revision is necessitated by the adoption, from October 2, by the building industry of a 42-hour week in place of a 44-hour week. The reckoner facilitates calculation by the formula $G = \frac{36W}{N}$, where G = the number of hours guaranteed, W = the number of working hours available, and N = the number of normal hours specified in the working-rule agreements.

Copies of the Standard are obtainable from the British Standards Institution, 2 Park Street, London, W.1; price 5s.

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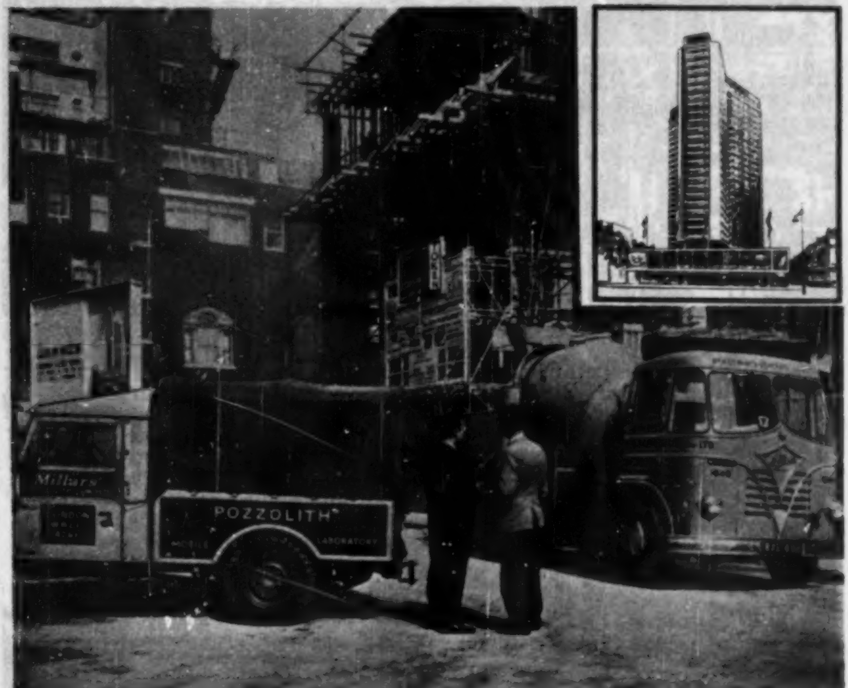

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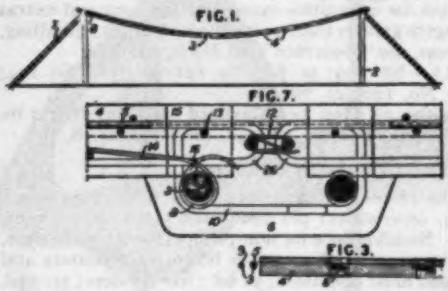
Patent Applications for Roofs.

Prestressed Roofs.

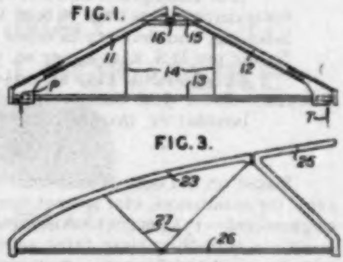
A PRESTRESSED concrete roof is formed by supporting reinforced concrete slabs 4 on cables or like tension members 3 suspended from and extending between supports 2, 7, 8 (*Fig.* 1), filling joints between slabs with mortar after all the slabs have been positioned, allowing the mortar to harden to produce a rigid roof unit, and then prestressing the concrete by tensioning the cables. The roof is assembled by laying the slabs at the lowest point and continuing uniformly on both sides, the slope at right angles to the cables being adjusted by suitable tensioning of cables 3.



In one embodiment, *Fig.* 3, the slabs 4 have reinforcing members 5 which project as hooks for engaging over the cables 3 which are enclosed in sheaths 9. The joints are filled with mortar 6; when this has set the cables are further tensioned to prestress the roof, and the cavities of sheathing tubes 9 are then filled with mortar under pressure. In another embodiment, *Fig.* 7, the slabs have projections or tongues 15 with grooves 16 for seating on cables 3 to which they are secured by bent reinforcing members 14. Hairpin loops 26 of reinforcement projecting from adjacent slabs are bound together by wires 12, and stirrups 10 are engaged around adjacent cables 3 and slab reinforcements 13. The slab ends at right angles to members 3 have quadrant shaped recesses filled with concrete and containing tensioning members.—No. 851,380. E. Lubbert, E. Schulz, U. Finsterwalder, R. Jecht, and E. Ruf. December 2, 1957.

Roof Trusses.

A JOINT between a lower end of an inclined reinforced or prestressed concrete rafter 11, *Fig.* 1, a tie bar 13 and a support for the rafter wherein the point of intersection P of the line of thrust T of the support with the axis of the tie bar is located below the extended neutral axis of the central part of the rafter, and in which the resultant of the forces exerted on the rafter 11 by the tie bar 13 and support exerts on the rafter a bending moment in the direction tending to bow the rafter upwardly between its ends thereby effecting reduction of the bending moment at the centre of the rafter. The rafter 11 may be part of a truss which may include a second rafter 12 meeting the rafter 11 in a ridge joint 16, which may comprise a



resilient packing, and which is located below the intersection of the extended neutral axes of the central parts of the two rafters. The ridge joint also includes at least one tie bar 15 which may be secured by nuts and washers on its ends, *Fig.* 5 (not shown). The invention may also be applied to a north-light truss, *Fig.* 3, in which additional reduction of bending moment is effected by an extension 25 on the shorter rafter 25, substantially in line with the longer rafter 23. The tie bars 13, 26 are supported by hangers 14, 27 respectively.—No. 861,676. G. Isaac. February 28, 1957.

Handbooks Received.

"The Federation of Civil Engineering Contractors Handbook 1961-62." (London: 1961. Price 12*s*. 6*d*.)

"Civil Engineering Films and Models." List of 111 films and fourteen models available from members of the Federation of Civil Engineering Contractors. Copies free from the Federation, Romney House, Tufton Street, London, S.W.1.

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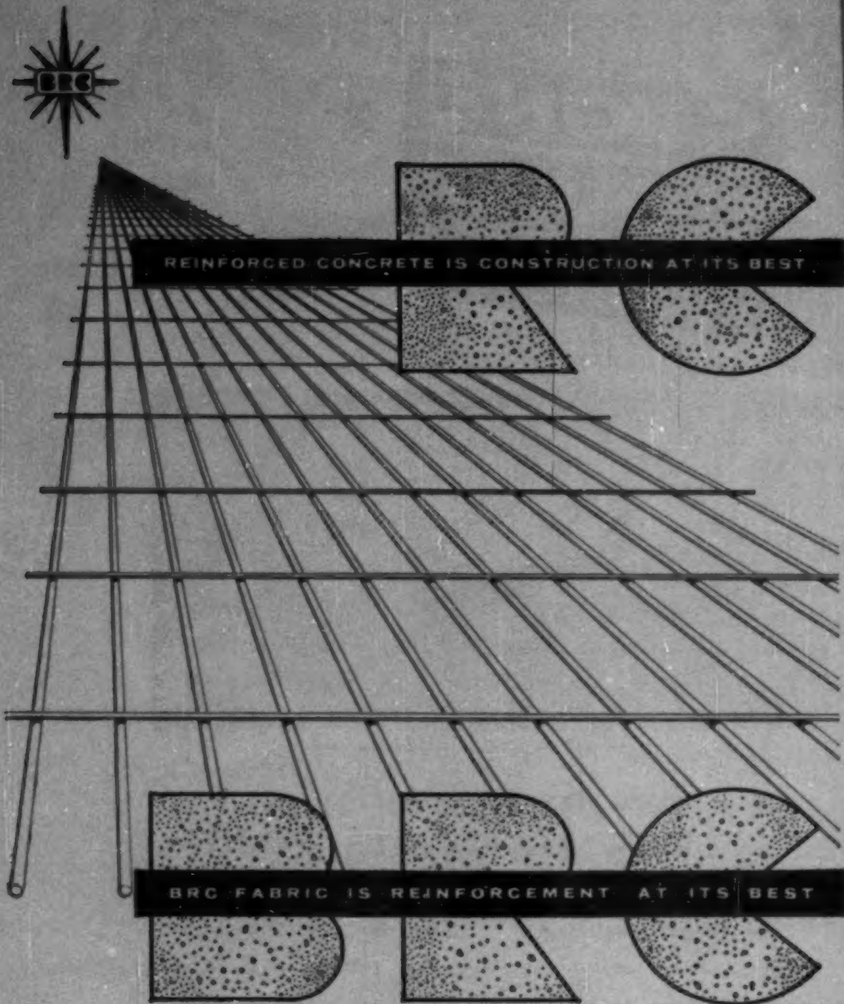

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